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SETTLEMENT EVALUATIONS FOR
FOOTINGS ON FINE SANDS

A THESIS


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FOOTINGS ON FINE SAND

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SUMMARY

Relatively little research in soils mechanics has been directed toward exploring the phenomena of settlements occurring in sands under load. In this study three possible methods of evaluating settlements in one type of sand are analyzed. The material studied is a uniform graded fine sand. The particle shape is subrounded and the soil is classified SP or A-3. This type of soil is naturally deposited along the coastal regions in most of the Southeastern states and, therefore, is commonly encountered in building construction.

The specific purpose of the study is to experiment with methods of predicting settlements by using both laboratory and load test derived indices. Load tests were conducted on prepared soils which were compacted under controlled conditions for evaluation of the analytical studies.

Three different analytical methods were studied. The first analysis was by means of simultaneous equations suggested by Hough (1) for settlement computations. These equations consider the soil's properties of compressibility, unit weight, size of the loaded area and other relevant factors that may influence settlement. The equations proved to yield invalid results. Through trial and error procedures one of the equations was modified. The new form of the

equations produced good accuracy within the range of conditions tested.

The second study considered the use of laboratory oedometers to define the soil's compressibility characteristics. This facet of the investigation did not prove fruitful. Numerous practical difficulties exist in preparing cohesionless soils for oedometer tests and the data derived did not yield accurate settlement information and the results were difficult to duplicate.

The last analytical concept consisted of exploring the use of a simple expression for elastic deformation. Although the settlements of the sands involved more than elastic deformation, the equation was found to yield a fairly good index to estimating actual settlement occurring beneath a larger plate.

A series of plate load tests were also run on the soils under conditions of 100 per cent saturation. High magnitude settlements were recorded for all these tests. Upon analysis the settlements at 100 per cent saturation were found to be associated with shear failure and, therefore, not relevant to the objective of this study. The data from these tests indicate the need for additional study on the rate of loading as an effect on the shear strength of saturated sands.

CHAPTER I

INTRODUCTION

General

During the development of soil mechanics as a science, many of the engineering properties of sands or cohesionless soils have been investigated in detail. From the findings of both theoretical and applied research, engineers now have access to a body of knowledge that will enable realistic estimates of most of the properties of sands as a building material. The vast majority of research, however, has been directed toward a resolution of the strength of sands from which have originated many of the concepts of "plastic theory" as applied in soil mechanics. These studies are invaluable to the foundation engineer charged with the responsibility of designing a footing system with the proper balance between safety and economy.

In contrast, relatively little work has been focused on the deformation and densification properties of sands as related to foundation settlements. In soft cohesive soils the settlement potentials are generally greater than in sands and much longer periods of time are required for settlements to develop. Engineers and scientists have accordingly developed and combined the theories of elasticity

and consolidation as an analytical tool. As a result, reliable mathematical models can be constructed for cohesive soils that provide sufficient accuracy of settlement estimates on clays for practical application.

On the other hand, settlements of footings on sands are seldom evaluated quantitatively by the practicing engineer. Although the structural safety of a building is usually not jeopardized by this omission, there are all too often unsightly telltale cracks as an aftermath. This lends credence to the need for additional studies of the elastic and densification behavior of sands under load as applied to foundation engineering.

The purpose of this research was to engage in a study of a narrow segment of the phenomena of sand settlements occurring under load. The work was limited to compacted sands of a homogeneous character. A series of plate load tests were conducted on the sands at varying controlled densities. An attempt was then made to analyze means of predicting settlements occurring beneath load test plates of varying sizes. Both laboratory test and field load test data were used as indices to settlement evaluation.

Previous Studies

Numerous authors through research and theoretical studies have contributed to a fuller understanding of the mechanics of soil deformation under load. A recent

treatise on the subject by Hough (2) served as the basis for much of the work undertaken in this thesis. His paper was presented as a general approach for achieving uniformity of footing settlements under varying loads rather than a specific study of the problem of sand settlement.

Hough suggested an application to sands and included probable indices that would be applicable to various densities of sands. In particular, he developed a relation between settlements and the "depth of significant stress". The "depth of significant stress" is described in general terms as the depth beneath a loaded area (footing) in which appreciable incremental pressures are imposed on the soils in excess of the unloaded or in situ stress condition. The "depth of significant stress" is developed as a function of three factors--the pressure at the base of the footing, the size of the loaded area and the stress history (including unit weight) of the soils. The concept proposed by Hough as related to this study is amplified in the Discussions and Appendix sections of this thesis.

The phenomenon of deformation or settlement of sands as a purely elastic material has been theoretically studied by several authors. Discussions of Barkan (3) on this subject were beneficial in the development of this work. He presented an equation expressing elastic settlements as a function of four factors. The four factors are the soil's modulus of elasticity, the Poisson's ratio, the area under

load and the pressures to which the soils are subjected. The application of Barkan's findings will be further developed in the Discussions section.

Laboratory one-dimensional oedometer tests have long been the established means of predicting the consolidation characteristics of clays. A (floating-ring) oedometer is one type of device that is widely accepted for this purpose. The general procedures for testing are included in the text by Sowers (4).

The extension of such tests for sands is not commonly undertaken. However, its possible application was explored in a series of oedometer tests. The general procedures of loading the sample and recording soil settlements were the same as used for clays. Sample preparation procedures were modified as needed for application to sands. These modifications are discussed in Chapter III under Oedometer Tests.

Other approaches have been proposed for estimating settlements on sand supported footings including semi-empirical relations which consider the soil's standard penetration resistance or cone penetration resistance. These studies were made largely by Terzaghi and Peck (5); however, their application is not analyzed in this study.

CHAPTER II

MATERIALS AND TEST EQUIPMENT

Soil

The analyses and tests undertaken in this study utilize one type of soil--a uniform slightly silty fine sand. This material was obtained from Jacksonville, Florida, and is typical of the upper mantle of soil which is found throughout much of the coastal areas of Southeastern Georgia, the outer perimeter of Florida, and along the seaboard regions of the Carolinas.

These fine sands are of "recent" geological origin and have been deposited in their present form since the Pleistocene or glacial epoch. Geologists indicate that these typical fine sands were progressively deposited along the advancing shore lines in a coastal environment as terrestrial uplift or buildup occurred.

The sands in their natural form are commonly loose or very loose near ground surface with a general increase in density with depth. Natural stratification of the sands is, of course, varied, dependent upon the conditions of deposition and secondary influences since that time. A generalized soil profile, representative of many typical findings is as follows:

<u>Depth (feet)</u>	<u>Soil Description</u>	<u>Normal Range of Penetration Resistance (ASTM D-1586-64T)</u>
0 -10.0'	Very loose to loose slightly silty fine sand	4 - 10
10.0-25.0'	Firm to dense slightly silty fine sand	10 - 40
25.0-35.0'	Firm to dense clayey fine sand	20 - 40
35.0'+	Beginning of calcareous marine deposit	

The descriptive properties of the soil under study and its grain size distribution are given in Figure 1. Also included in Figure 1 are the soil's classification symbols by both the Unified and the Bureau of Public Roads systems.

Test Equipment

The moisture-density relationship was determined by using the "Modified Proctor" compaction equipment and procedures, ASTM D-1557. The equipment consists of a 1/30 cubic foot mold and a 10 pound hammer with an 18 inch fall.

The soils in the test pit were compacted to the pre-selected density (a uniform percentage of the maximum density) by using two devices. A manual hand tamper was used to obtain 85 per cent of the "Modified Proctor" maximum. A motorized compactor was used for the higher densities of 90 and 95 per cent.

The hand tamper was constructed from a 6 by 6 inch

timber with a 10-inch square base plate nailed to the bottom. Handles were attached to facilitate manual use of the device. The tamper weighed approximately 15 pounds.

The soils at 90 per cent of maximum density and higher were compacted using a manually guided vibratory sled compactor (Wacker, Model VPG-160). This compactor was equipped with its standard face plate with dimensions 20 by 20 inches. The vibratory frequency of the compactor is 5000 cycles per minute with a rated impact of 1700 pounds.

The density of the soil in the test pit was controlled from the results of sand cone density tests. An Ottawa sand grading No. 20-30 U. S. standard sieve was used in the density apparatus and calibrated for the specific six-inch cone and base plate used in testing. The equipment and techniques of density testing are in accordance with ASTM D-1556-58T.

The load test plates were fabricated from plate steel with thicknesses to provide rigidity in accordance with ASTM standard D-1194-57. Three plate sizes were used in the experiment--a one square foot plate, a two square foot plate and a four square foot plate. The incremental loads were applied to the plates by means of a 10-ton capacity Blackhawk jack, Model No. B 107005 calibrated to within 1 per cent. The settlements occurring under each load increment were measured to the nearest .001 inch by three micrometer dial gauges. The micrometer dial gauges were affixed to a

reference bar perpendicular to and separate from the load truss. An anchored aluminum truss served as the reaction for the jacking load, and swivel joints were provided both at the base of the test plate and at the bottom of the loading truss to allow freedom of rotation of the plate.

Laboratory tests were conducted on the soils using standard equipment. The oedometers used in testing were of the "floating ring" type, 2.5 inches in diameter and one inch thick. The incremental loads were applied to the soils in the oedometer by a direct rather than a lever system to provide a closer interval of loadings than would be available using the regular weights and conventional lever ratios.

The triaxial cell utilized a 1.4 inch diameter, three inch high soil sample. An internal vacuum was used to provide confinement in some cases and external air pressure in others with control by means of a sensitive pressure regulator. Incremental loads were applied to the sand column by a manually operated worm-gear platform connected to a Fairbanks-Morse scale.

CHAPTER III

TESTING PROCEDURES

General Comments

The most important aspects of the experimental procedures included the establishment of a pre-selected and controlled sand density in the test pit, and secondly, the development of a suitable method for oedometer testing of sand. Numerous other necessary or supplementary tests were conducted during the course of the experiment, such as moisture-density tests, field density tests, grain size tests, triaxial shear tests. These were all performed in accordance with prevailing standards. For this reason, the major portion of this section will apply to non-standard features of the experimental work. A descriptive resume is included for other tests with reference made to the appropriate testing standard.

Test Pit

The test pit was enclosed in a wood frame structure with a roof and siding. The shelter provided by the enclosed working area prevented excessive moisture variations in the fill from either wetting or drying and, of course, protected the completed fill from damage by rainstorm.

The pit occupied an area 12 by 14 feet. The soils

were hand excavated to a depth of 5.5 feet. The relative layout of the test pit is shown on Figure 2. A cross section also shows the six-inch gravel layer installed in the bottom of the pit to facilitate obtaining 100 per cent saturation of the test soils. Also, polyethylene sheeting was placed around the sides of the excavation to minimize lateral seepage. A four-inch diameter concrete pipe was installed in each corner of the test pit from the gravel layer to ground surface. These concrete pipe sections provided for both in-flow and monitoring of the saturation water.

Compaction and Load Test Sequence

Eleven load tests were conducted in the experiment with the soils in the pit compacted to three different density levels. Each test was conducted approximately two inches below the top surface of the pit.

The first sequence of load tests were on soils compacted in the pit to a target density of 85 per cent of the Modified Proctor maximum. The pit was filled with consecutive loose layer thicknesses of about six inches. Each layer was compacted by the hand tamper after which a sand cone density test was conducted to confirm the attainment of the desired level of compaction. Figure 3 demonstrates the variation in density from the start of the filling operation to its completion. The average density actually obtained in the completed pit was 85.5 per cent of maximum. Emphasis

was placed on obtaining uniform compaction because a homogeneous soil was a necessary prerequisite for valid testing.

The second and third sequences of load testing were performed with the soils compacted to approximately 90 per cent and 95 per cent of the maximum density, respectively. The soils at these two higher density levels were also compacted in six inch layers except that the vibratory sled was used to densify each layer. One pass was normally sufficient to produce 90 per cent compaction while two to four passes were needed to produce 95 per cent compaction. Figures 4 and 5 represent the variation of density in the test pit at the 90 and 95 per cent density levels.

Four load tests were conducted on the sand at 85 per cent compaction. A similar series of four tests were run at 90 per cent compaction, while at 95 per cent compaction three tests were run in a modified sequence.

One purpose of the load test was to relate the size of the loaded area (plate size) to the magnitude of settlement. Two plate sizes were used in each series of tests at 85 and 90 per cent compaction. The area of the small plate was one square foot and the larger, two square feet. The testing area was divided into four quarters with load test designated to be conducted in the center of each quarter area of the pit.

The first test was on the one square foot plate with the soils at the compacted moisture content of 4 to 6 per

cent. The second test was conducted using the two square foot plate size with the same moisture conditions.

After the first two tests, the pit was 100 per cent saturated with water and the one and two square foot plate size tests repeated in the remaining two quarters of the pit.

The water level was controlled during the entire test by periodic monitoring of the water rise in three of the concrete pipes at the corners of the pit. The inflow water was introduced through the remaining pipe. The rate of inflow, after saturation, was adjusted to provide equilibrium with the loss occurring from lateral and vertical seepage. The seepage loss was estimated at three gallons per minute. During the 100 per cent saturated tests, the water was maintained virtually at ground surface and the hydrostatic head variation was held within a one-inch range as measured at the three monitoring pipes. In this manner hydrostatic pressures at the plate locations were virtually constant.

The load test at 95 per cent compaction did not include any at the saturated condition. The saturated tests were eliminated from these series because of questionable results as will be explained later in the Discussion section. Instead, the tests at this higher density include a one square foot plate, a two square foot plate and a larger four square foot plate to complete the sequence.

The results of the plate load tests are presented in the form of load-settlement curves. The tests results

at 85 per cent compaction are shown on Figure 6. The load-settlement curves at 90 and 95 per cent compaction are shown on Figures 7 and 8.

Oedometer Tests

Since the test soil was a cohesionless sand, conventional procedures ordinarily used for preparation and testing of the settlement characteristics of clay in an oedometer could not be used. The purpose of the oedometer tests was to compare the predicted soil settlements obtained by this means to the actual settlements developing in the test pit. For this reason it was imperative to have the soils in the oedometer at the exact structures (expressed in terms of density) as those in the test pit and it was this problem of sample preparation that was most difficult to achieve. Once the proper sample was made up and installed in the oedometer the test could be completed using more or less standardized procedures.

After experimenting with several different methods of sample preparation, the most reliable was adopted and used for completing this series of tests. The first step consisted of accurately weighing a calculated amount of oven-dry material to produce the desired per cent compactions as set up in the test pit. Approximately six to eight milliliters of water was then added to the soil and thoroughly mixed. The added water produced moisture contents in the

order of eight per cent. This amount of moisture was needed to develop sufficient capillary tension to keep the sample intact while being inserted into the oedometer.

This prepared soil was actually compacted in about 1/8 inch layers into the oedometer ring. After several attempts, the balanced amount of compactive effort could be achieved by which the soils would exactly fill the one-inch high ring and thus provide the desired density. The ring, together with the soil, was then carefully inserted into the oedometer, sandwiched between porous plates and made ready for incremental loading. The settlements under each load were measured by a micrometer dial gauge.

Once the test was set up, the incremental loads could be applied in close sequence because the settlements occurred instantaneously during loading, and there was little or no continuing (consolidation type) deflection noted. Nevertheless, each increment was held for a total of ten minutes.

The results of the oedometer tests at 85 per cent and 90 per cent compactions are shown on Figures 9 and 10.

Grain Size Tests

The grain size distribution of the soils were determined by mechanical analysis in accordance with the provisions of ASTM D-421-58 and D-422-54-T. In this test the soil was passed through a set of interlocking sieves and the weights retained on each sieve measured for computation

of the per cent finer by weight. The portion retained on the 200 mesh sieve was washed to remove smaller adhering particles.

Maximum and Minimum Density Tests

The maximum and minimum density of the test sample were determined for evaluating the relative density of the in-place fill soils. The maximum density of the soil was taken as a "Modified Proctor" maximum in accordance with ASTM D-1557. The soils were compacted in five equal layers with each layer receiving 25 blows from a 10-pound hammer falling 18 inches.

The minimum density of the test soils was determined by funneling a sample of dry soil into a mold of known volume. The funnel was kept in the center of the mold and the mouth of the funnel just above the level of the rising cone of soil. Test results are recorded on Figure 11.

Triaxial Shear Test

Several triaxial tests were conducted on representative samples of the test soil. In the various tests the confining pressures were maintained either by vacuum or by positive confining pressure methods. The vacuum method is normally used for shear testing of cohesionless soils. The sample was prepared at three separate densities varying from loose to firm. Each sample, in turn, was placed into the triaxial cell and an axial load applied until the soil

failed in shear. The relation between the soil's density and angle of internal friction was developed from the test data. The graphs depicting this relation are included in Figures 12 and 13 which also indicate the Mohr envelope at the various density levels.

Additional triaxial shear tests were also conducted at pre-established densities with the confinement provided by applying positive pressure around the sample in the triaxial cell. The results of all triaxial shear tests are shown in Figures 14 and 15.

Penetration Tests

After completing the load tests at 95 per cent compaction, a dry sample boring was made in the pit to determine the soil's penetration resistance. Sampling and testing were performed in accordance with the technical provisions of ASTM D1586-64T.

The boring was advanced by means of a hand auger. The penetration test was made at regular intervals throughout the depth of fill. The sampler was first seated six inches to penetrate any loose cuttings and then driven an additional 12 inches by blows of a 140 pound hammer falling 30 inches. The number of hammer blows required to drive the sampler the final foot is equal to the soil's penetration resistance. A log showing the penetration test data is depicted on Figure 15.

CHAPTER IV

EVALUATION OF TEST RESULTS

The experiments afforded two categories of information related to load testing of sands. The purpose of the work and primary category of data is relevant to load-deflection measurements and settlement predictions. The second category of information applies to soil strength or shear phenomenon. This aspect of the paper is incidental and developed as a result of the shear failures occurring in the load tests which were conducted at 100 per cent saturation. The shear failures in these tests produced excessive plate deflections which could not be referenced to normal settlements occurring without shear failure.

The problems encountered in the load tests conducted at 100 per cent saturation will be evaluated in a later paragraph of the Discussion section. The related soil strength observations will also be presented.

The major subject matter of this section, however, analyzes the observed plate settlement findings as related to three trial methods for predicting settlements by experimental and analytical means. The first and primary section discusses the approach presented by Hough for settlement predictions. In the second section the oedometer studies are

discussed and in the last section the test findings are related to an elastic deflection equation from Barkan (6) which is developed from the original Boussinesq theory.

Load Test Findings as Related to Hough's Equation

The method proposed by Hough for evaluating and equalizing settlements in a foundation system were studied in detail when planning the experiment. In brief, Hough developed a mathematical expression which relates settlements to the primary variables that influence settlements. These variables include the footing pressure, the size of the loaded area, the stress history of the soil (overburden and unit weight), and the consolidation characteristics of the material under load. The derivation of these relations is included in the Appendix. The mathematical expressions applicable to this work, however, are as follows:

$$h_s (h_s + B)^2 = \frac{10 B^2}{\gamma} p_c$$

$$\frac{10}{27} h_s \left(10^{\frac{SC}{h_s} - 1} \right) (h_s + 3B)^2 = \frac{10 B^2}{\gamma} p_c$$

(Symbol identification is found in the Appendix)

These two simultaneous equations are for footings at ground surface where the plate load tests of this experiment were conducted.

The objective of this phase of the experiment was to

evaluate the use of these equations for predicting settlements as influenced by different size foundations. To explore this objective the following analytical procedures were used. First, the equations were solved using the load test data from the one-foot square plate. The load test provided known values for contact pressure ' p_c ', settlement ' s ', unit weight of soil ' γ ' along with the footing width ' B '. The two unknowns, depth of significant stress ' h_s ' and the bearing capacity index ' C ' were determined for the one-foot plate by solving the pair of equations. The derived ' C ' value, thus obtained, was used for computing the settlements beneath the larger two square foot or four square foot plate at the same soil pressure. The predicted settlements derived by solution of the equation were then compared to the actual settlements determined by load testing at the larger plate sizes. The results of the comparisons are shown on Figure 17 in graph form. In this graph numerical values are avoided. Instead, the ratio of settlement of the larger to the smaller plate is plotted versus the ratio of the width of the larger to the width of the smaller plate.

As may be seen, the settlements computed from Hough's relation for the larger footings do not agree with the observed test results. In ratio of footing size to settlement, the computed values are approximately 40 per cent low. This means that in actuality the effect of the footing size is much more pronounced than the computed results indicate. In

their present form, therefore, the equations do not seem suitable as an analytical device for developing load-settlement relations between footings of varying sizes that are supported on fine sands of a uniform character.

At first it appeared that the poor relationship between the computed and actual findings was attributable to the mathematical expression used for decrement of stress with depth as incorporated in the derivation of the equation. In Hough's equation, the incremental body stress for a square footing is expressed by:

$$\Delta p = \frac{B^2}{(h + B)^2} p_c$$

This expression obtains an incremental pressure over a zone defined by planes at an angle 63.5 degrees from the horizontal at the edge of the footing.

New equations were then derived by the writer using a modified pressure--depth expression as follows:

$$\Delta p = \frac{B^2}{\left(\frac{h}{2} + B\right)^2} p_c$$

In this expression the "pressure zone" is inscribed by an angle 76 degrees from the horizontal at the footing edges. The primary equations derived using the above modified

pressure relation are as follows:

$$h_s (h_s + 2B)^2 = \frac{40 B^2}{\gamma} p_c$$

$$\frac{40}{108} h_s \left(10^{\frac{5C}{h_s}} - 1 \right) (h_s + 6B)^2 = \frac{40 B^2}{\gamma} p_c$$

The new equations, however, yielded identical results (within slide rule accuracy) to that of the original equations. A second pair of equations were then derived in a trial solution where the pressure zone was defined by an angle 45 degrees from the horizontal at the edge of the footings. This pressure relation yielded the following basic equations:

$$h_s (2h_s + B)^2 = \frac{10 B^2}{\gamma} p_c$$

$$\frac{10}{27} h_s \left(10^{\frac{5C}{h_s}} - 1 \right) (2h_s + 3B)^2 = \frac{10 B^2}{\gamma} p_c$$

This new form also produced computed settlements that were identical to the original equations. This clearly indicated that the invalidity of computed settlements did not result from the pressure-decrement expression.

A third modification of the original equation was then undertaken. In Hough's original derivation, the average incremental pressure ' p_c ' was assumed to occur at one-third of the effective depth ' h_s '. Instead of computing this pressure at one-third ' h_s ' the writer derived new equations which

express the average incremental pressure 'p' at one-fourth 'h_s'. With the above modification the equations are in the following form:

$$h_s (h_s + B)^2 = \frac{10 B^2}{\gamma} p_c$$

$$\frac{h_s}{6.4} \left(10^{\frac{SC}{h_s}} - 1 \right) (h_s + 4 B)^2 = \frac{10 B^2}{\gamma} p_c$$

Upon solving for settlements, however, it was found as before, that the relationship of settlement to footing size remained unchanged.

After the above unsuccessful attempts at obtaining a workable expression for settlements, it became obvious that the difficulty lay in the exponential expression for the compression ratio. In Hough's derivation the bearing capacity index 'C' was expressed as:

$$C = \frac{1 + e}{C_c}$$

In this expression the compression index 'C_c' was considered as a constant value from the void ratio versus log of pressure curve. Figure 18 illustrates the assumed conditions for compression index 'C_c'.

The oedometer studies (to be later discussed) reveal that 'C_c' is not constant for fine sands within the pressure

range used for testing. Instead, the compression index increased at higher pressures beyond the unloaded condition, a typical example of which is shown in Figure 18. It was therefore concluded that the larger footings which impose stresses into the soil to a greater depth were, in fact, operating at a point further along the void ratio versus pressure curve than the smaller footing at attendant higher compression indices. It, therefore, followed that a higher 'Cc' would be effective for the larger footing and in terms of Hough's equation, a smaller bearing capacity index, 'C', which is inversely proportional to 'Cc'.

The original equations followed a rational derivation but as indicated above, considered the bearing capacity index as a constant term. The actual variations in the bearing capacity index, in fact, presented an obstacle toward the effective use of the equations. To overcome this deficiency the writer developed an empirical modification of the basic equations in an attempt to more accurately express the observed variations in the bearing capacity index. The modification consisted of multiplying the 'C' term by the ratio $\frac{h_{s1}}{h_{s2}}$ where 'hs₁' is the effective depth of stress beneath the one square foot plate and 'hs₂' the effective depth of stress beneath the larger plate. The equations are now as follows:

$$h_s (h_s + B)^2 = \frac{10 B^2}{\gamma} p_c$$

$$\frac{10}{27} h_s \left[10 \left(\frac{h_{s1}}{h_{s2}} C \right) \frac{S}{h_{s2} - 1} \right] (h_s + 3B)^2 = \frac{10 B^2}{\gamma} p_c$$

With the above final modification, the equations produce computed settlements almost identical to the measured quantity. In fact, the errors in computed results were less than three per cent off observed values for all but one of the load tests. This is illustrated in the following tabulation:

<u>Per cent</u> <u>Compaction</u>	<u>Plate</u> <u>Size</u>	<u>Computed</u> <u>Bearing</u> <u>Capacity</u>	<u>Bearing</u> <u>Index</u>	<u>Bearing Capacity</u> <u>Index to Obtain</u> <u>Actual Settlement</u>	<u>Per Cent</u> <u>Error</u>
85	2	253		261	+3.1
90	2	612		625	+2.1
94.5	2	715		712	-0.4
94.5	4	575		518	-11.0

This close agreement may involve some measure of coincidence for, in actuality, it would be difficult to produce companion load tests with settlements varying less than 3 per cent for a given pressure. Nevertheless, the results confirm that the final revision of the equation may be used to accurately compute settlements within a given range of footing sizes for the soil under study. Actually, there are two other limiting conditions. In the strictest sense the empirical modification of Hough's equation is verified only

for fill sand such as used in the experiment, and secondly, the sand must be of uniform structure and density. The validity of the modified equation within the limited range of trial conditions is encouraging and suggest that a broader application may exist.

In Hough's paper a graph was prepared that suggested the probable relation between bearing capacity index and the standard penetration resistance for different types of soils. It appears, however, that his proposed relation of standard penetration resistance to bearing capacity index would need revising for fine sand of uniform gradation. After completing the test at 94.5 per cent compaction, a penetration test boring was made within the pit. This yielded an average penetration resistance of seven to eight blows per foot on the standard spoon. When compared to the curve for the most similar soil on Hough's graph, a bearing capacity index of about 60 is indicated. However, from the load test results and computations, it appears that the average bearing capacity index should be more in the order of 500 for this particular soil within penetration range of seven to eight blows per foot.

Load Test Findings as Related to Oedometer Studies

The computations of settlements from the oedometer tests results were set up in a similar manner to that used for computing settlements on clays. The soils beneath the

considered footing were divided into a number of finite layers for numerical integration of the settlements occurring in each layer. For each case the layers beneath the footing were considered to a depth comparable to a z/b ratio of approximately 3.5 (z equals vertical distance to layer; b equals footing width). The average vertical stresses were obtained from the Boussinesq equation. The incremental settlements occurred in each layer were computed using the relation for one-dimensional compression:

$$\Delta h = H \frac{\Delta e}{1 + e_0}$$

The appropriate values for void ratios used in the above expression were obtained from the oedometer test results in which the void ratio was plotted versus the log of vertical pressure.

The computed settlements using the oedometer test data were satisfactory in one respect and poor in another. A satisfactory agreement was obtained between the observed and computed footing size to settlement ratios. This is shown in Figure 19. A much poorer agreement, however, was obtained between the magnitude of computed settlements for a given plate size to the actual settlement occurring under the same pressure. In all cases, the computed settlements were considerably above the actual settlements as shown below:

<u>Per Cent Compaction</u>	<u>Plate Size</u>	<u>Pressure (psf)</u>	<u>Actual Settlement (inches)</u>	<u>Computed Settlement (inches)</u>	<u>Excess in Computed Results</u>
85	1	1000	.060	.0922	54 %
85	2	1000	.085	.1322	57 %
90	1	2000	.033	.0693	110 %
90	2	2000	.048	.0993	107 %

In each case as noted on the above tabulation the computed settlements using the oedometer results were in excess of 50 per cent of the actual observed settlement. From the results obtained in these studies, it does not appear that the oedometer is a suitable device for developing load-settlement or load-void ratio parameters for a sand. First, it is very difficult to control the initial densities in sample preparation. Several trial attempts were found to be necessary to uniformly place a pre-measured quantity of sand into the oedometer. Secondly, there may be inadvertent shocks or vibrations during loading which tend to over consolidate the sample as compared to the more static conditions of a plate load test.

A third condition which may effect the accuracy of testing is the moisture content variation between field and lab test soils. The moisture content in the load test soils were in the range of 3 to 5 per cent. For the oedometer test, however, it was necessary to prepare this particular sand at a moisture content of approximately 8 per cent to keep the soil intact during insertion into the oedometer.

It is quite likely that the difference between the negative pore pressures or capillary tension at 4 per cent and 8 per cent moisture would be sufficient to affect the pressure-void ratio relation. At 4 per cent moisture the sands would partially break down if molded to a ball shape by hand pressure. The higher capillary tension at 8 per cent moisture, however, was sufficient to retain the shape of a hand molded sample of sand. No attempt was made to delineate the effects of the negative pore pressure variable in this experiment.

Plate Settlements as Compared to An Elastic Equation

An equation is given in Barkan's book (7) for the elastic compression occurring in a soil of homogeneous character under load. The expression derived in this book (from Boussinesq theory) is as follows:

$$s = p \frac{(1 - \mu^2)}{1.13 E} \cdot \sqrt{A}$$

Where: p = contact pressure E = modulus of elasticity
 μ = Poisson's ratio A = footing area

For a given soil with constant values for pressure, Poisson's ratio and modulus of elasticity the above expression could be stated as a useful ratio in the form:

$$\frac{s_1}{s_2} = \frac{\sqrt{A_1}}{\sqrt{A_2}} = \frac{B_1}{B_2}$$

Where: B = footing width

The results of the plate load test are shown in Figure 20 as compared to the computed settlements using the above expression. As may be seen, the actual settlements are reasonably close to that obtained using the expression derived for elastic compression of a soil. In sands it is known that the value of modulus of elasticity and Poisson's ratio are not constant, but in fact vary with confinement and deviator stress. Also, in a loose sand some "pseudo" consolidation or densification takes place under load.

It appears, however, that the expression for elastic deformation may be useful as a simple and fairly accurate means of rating the settlements of larger footings to the results obtained from smaller scale load tests. For sands, at least of the type and within the range of conditions tested, most of the settlement occurring within the safe load range is predominantly elastic.

Plate Load Test With Sands 100 Per Cent Saturated

As mentioned in the introductory paragraph, the test at 100 per cent saturation did not provide useful settlement data for analytical comparison. Instead of a linear-load settlement relation, the test at 100 per cent saturation were evidently in a state of progressive shear failure with each incremental load as shown on Figures 6B and 7B.

The test loads were applied to the soils in increments of approximately 175 psf for 85 per cent compaction.

The incremental load was applied relatively quickly by jacking within a typical elapsed time of five seconds. Apparently this quick application of the load was sufficient to cause excessive pore pressure buildup in the sands and thereby induce a temporary drastic reduction in the soil's shear strength.

Although the 100 per cent saturation tests provided no useable settlement data, the test information is interesting from the standpoint of bearing capacity. As a check on soil's shear strength several triaxial tests were conducted on saturated sands. A drained test produced an angle of internal friction ' ϕ ' only slightly less than that obtained on a dry sample. A triaxial test conducted at undrained conditions (Figure 13) produced much lower ' ϕ ' angles especially at the higher void ratios as positive pore pressure developed upon densification.

The comparative results of the load test derived apparent ' ϕ ' angles and those obtained by laboratory testing are illustrated in the following illustration:

<u>Per Cent Compaction</u>	<u>by Triaxial Testing</u>	<u>Plate Size</u>	<u>Estimated ϕ from Load Test</u>	<u>Per Cent Reduction in ϕ Angle</u>
85	35	1	25	29
85	35	2	23	34
90	37	1	32	14
90	37	2	33	11

The computed apparent ' ϕ ' angles in the above tabulation were developed using the bearing capacity factors developed by Meyerhof (8) for shallow footings.

As was found in the laboratory tests, the greater loss of shear strength upon loading occurred in the looser of the two conditions where the added densification upon loading produced greater neutral or pore pressure. It is also important to note that the reduction in the bearing capacity factor is not lineally proportional to a reduction in the ' ϕ ' angle. In other words, a percentage reduction in the ' ϕ ' angle would cause a much higher reduction in the bearing capacity of a cohesionless soil near ground surface.

When designing footings which are to be placed in submerged or saturated sands, it is obvious that the engineer must exercise good judgment to avoid marginal bearing capacity or even failure. If quick loads are to be imposed upon loose sands during or shortly after construction there may be severe reduction in bearing capacity. Additional study would be needed to provide indices for estimating the rate of loading that would be satisfactory for different types of soil.

CHAPTER V

CONCLUSIONS

The following conclusions have been reached as a result of this study:

1. Plate load test data on sands may be used for estimating the settlements of larger size footings.
2. The relationship of the square root of footing area to settlement provides the simplest means of obtaining approximate estimates of footing settlements from plate load test data.
3. The equations developed by Hough do not appear suited for computing footing settlements from plate load test data.
4. The difficulty in applying Hough's equations probably stems from an assumed constant value for the compressibility index.
 - a. Oedometer tests reveal that the compression index for the investigated sands is not constant.
5. A modified form of Hough's equations may be used for more accurate relations between settlements and footing size.
6. Laboratory oedometer tests are tedious to perform on cohesionless sands and the results may be misleading for settlement computations.

7. The rate of load application to water saturated footings is an important consideration.
 - a. Quick loading under saturated conditions may cause severe reduction in the soil's shear strengths.
 - b. Settlements will be increased with footings in a saturated condition.

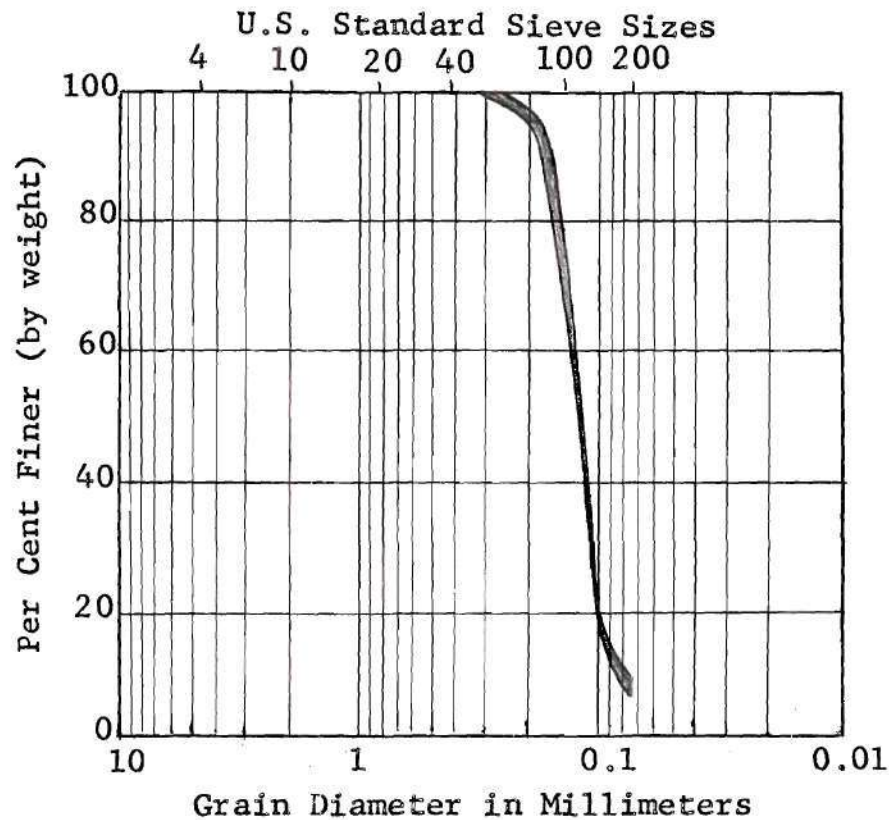
CHAPTER VI

RECOMMENDATIONS FOR FURTHER STUDY

The following recommendations are made for further study:

1. Conduct load tests on soils using a wider range of plate sizes.
2. Explore the validity of the developed equations (modified from Hough) over a wider range of plate sizes and soil conditions.
3. Evaluate the use of the developed equations for footings of different configuration and at different depths.
4. Conduct experimental load tests on saturated soils with controlled rates in load application.
5. Develop criteria for evaluating the influence of loading rate on shear strength in saturated sands.
6. Review the possibility of using triaxial shear test data for evaluating the settlement potential of sands under load.

APPENDIX



Soil Description - Grey slightly silty fine sand

Unified Classification - SP

Bureau Public Roads Classification - A-3

Plasticity Index - Non Plastic

Specific Gravity of Solids - 2.648

Grain Shape - Subrounded

Uniformity Coefficient - 1.6

Figure 1 - Descriptive Properties of Test Soils

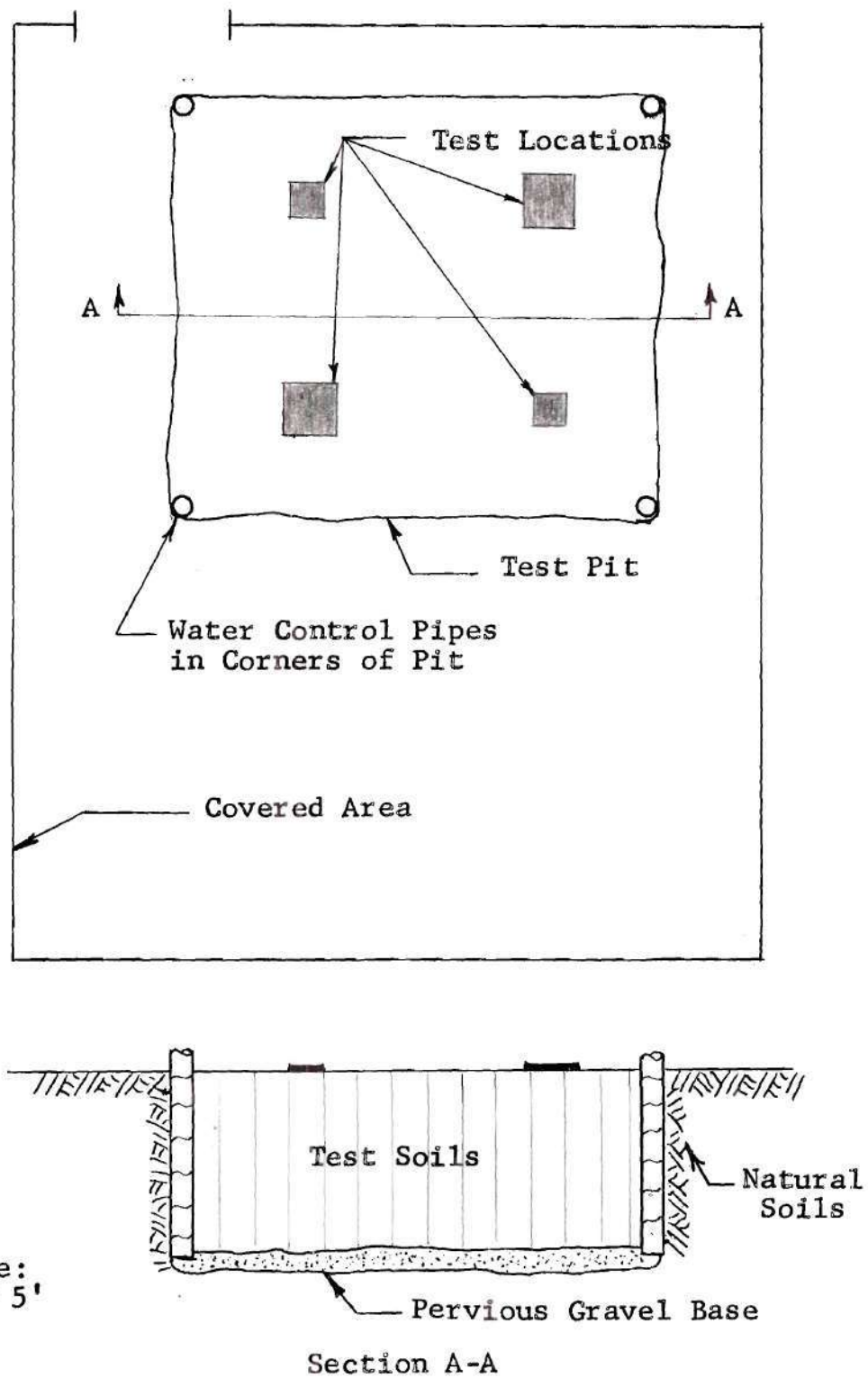


Figure 2 - Layout Of Test Pit

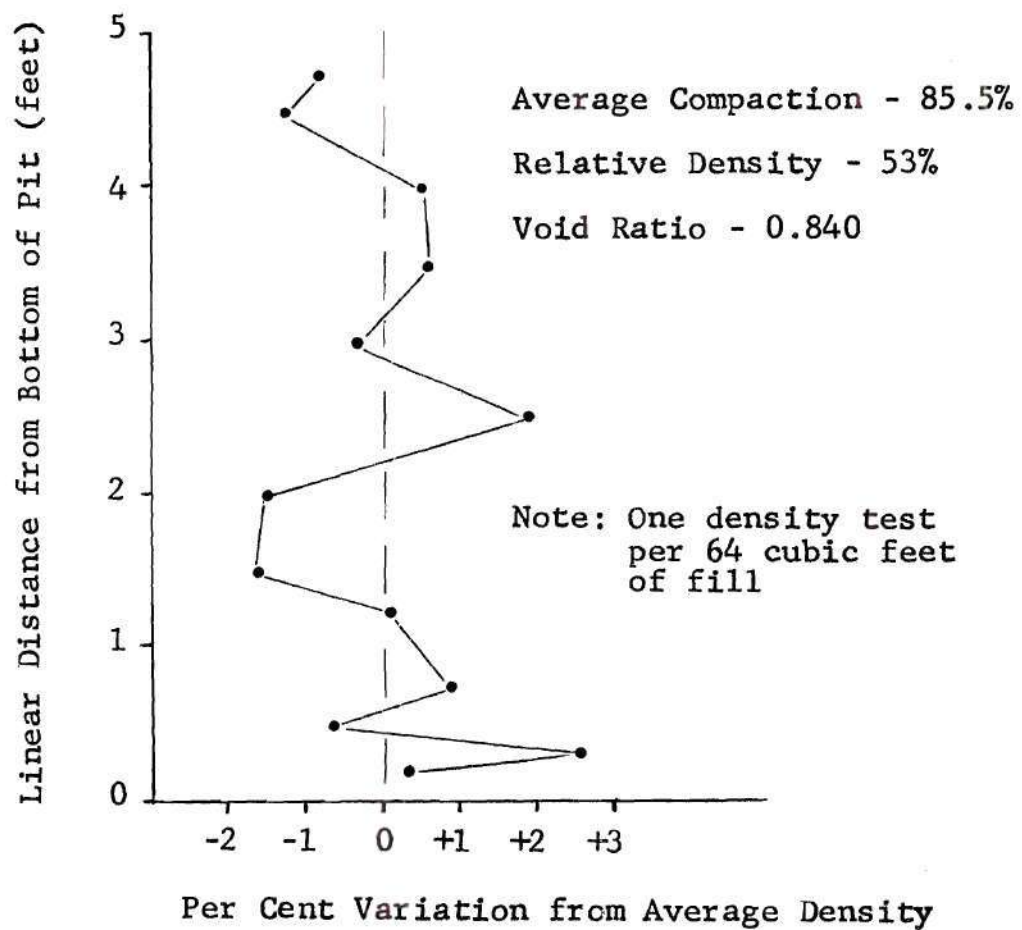


Figure 3 - Density Variations in Test Pit
at 85 Per Cent Compaction

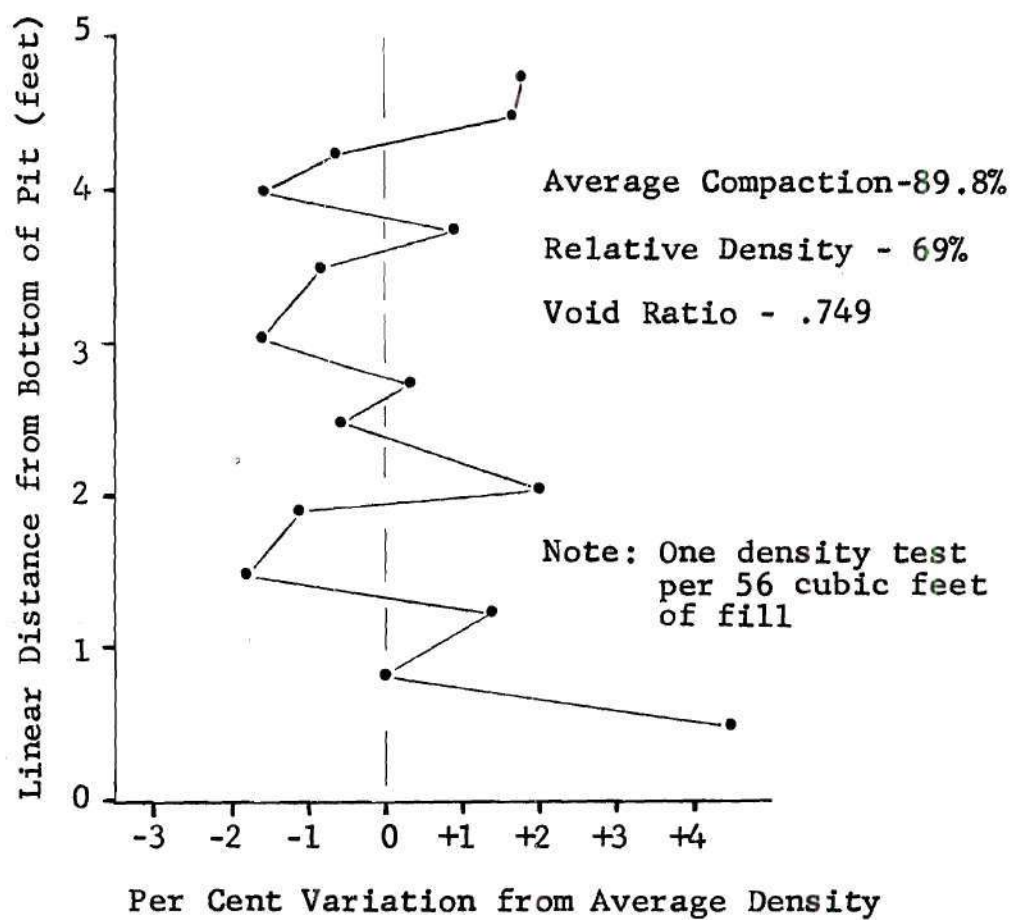


Figure 4 - Density Variations in Test Pit
at 90 Per Cent Compaction

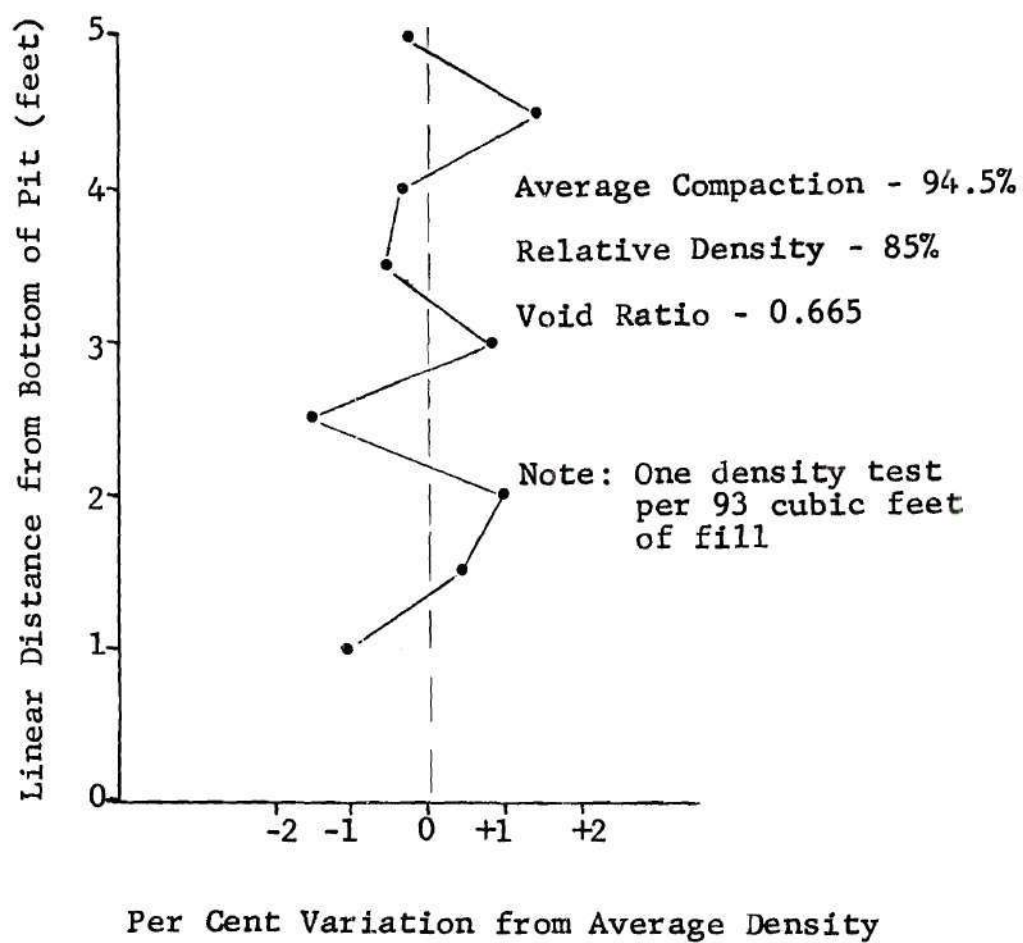


Figure 5 - Density Variations in Test Pit
at 95 Per Cent Compaction

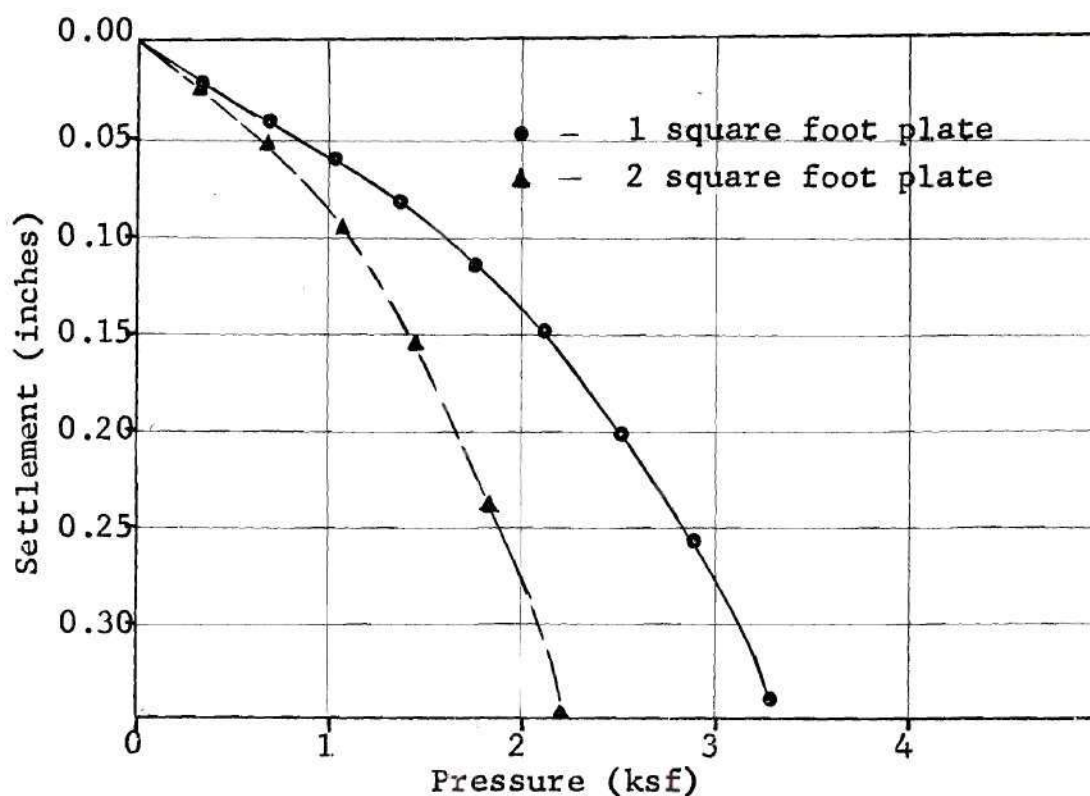


Figure 6A - Plate Settlements, Pit at 85 Per Cent Compaction

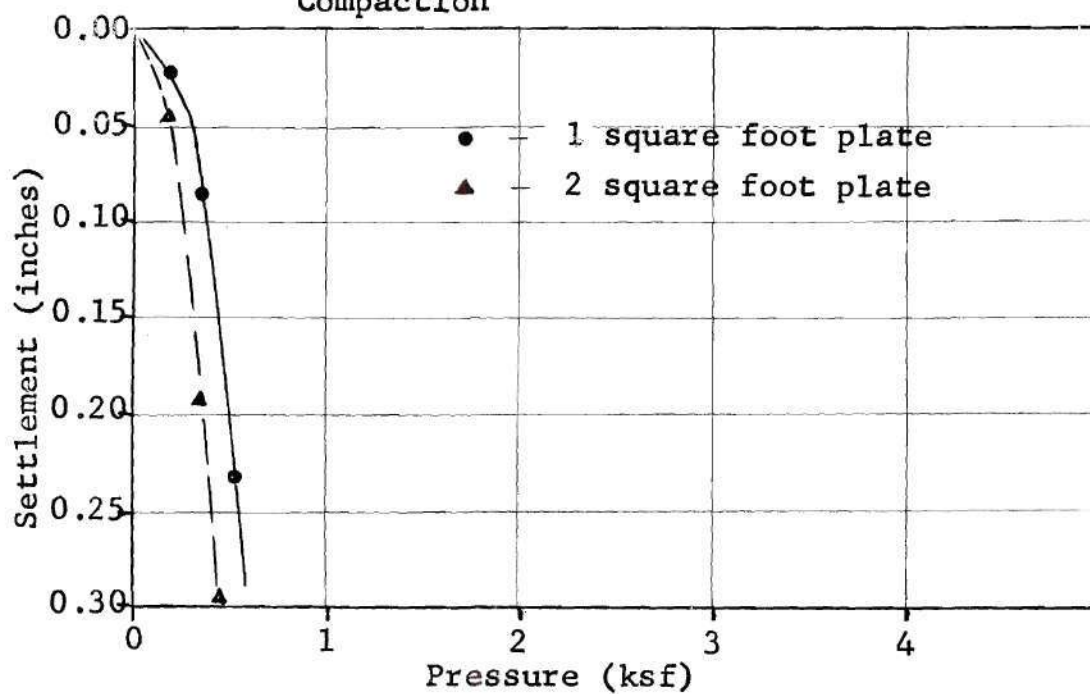


Figure 6B - Plate Settlements, Pit at 85 Per Cent Compaction - Flooded Condition

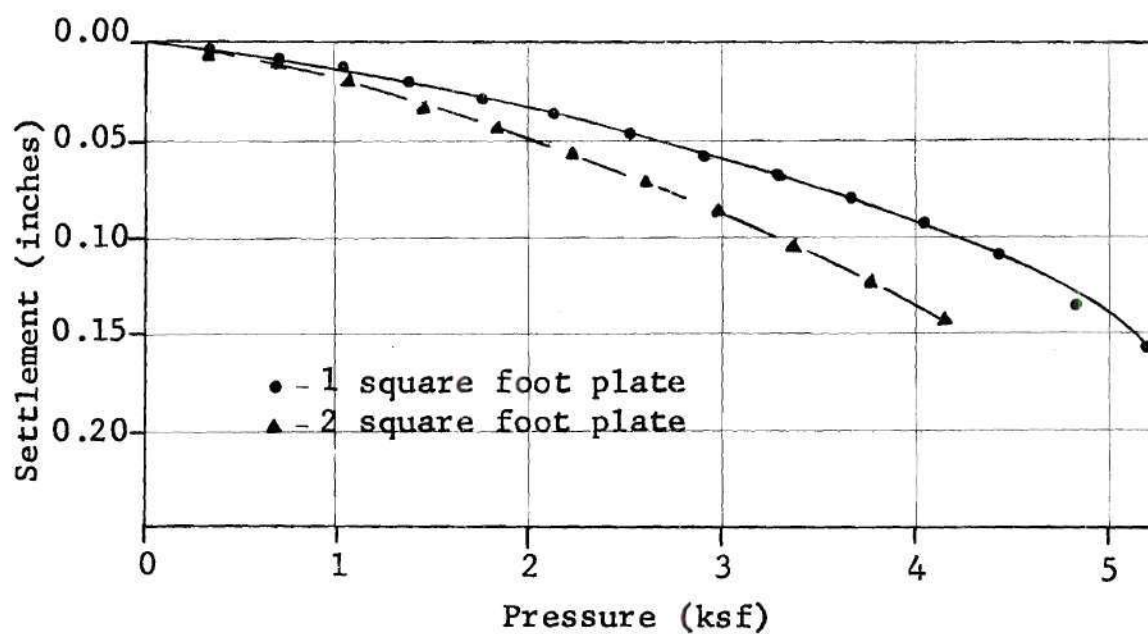


Figure 7A - Plate Settlements, Pit at 90 Per Cent Compaction

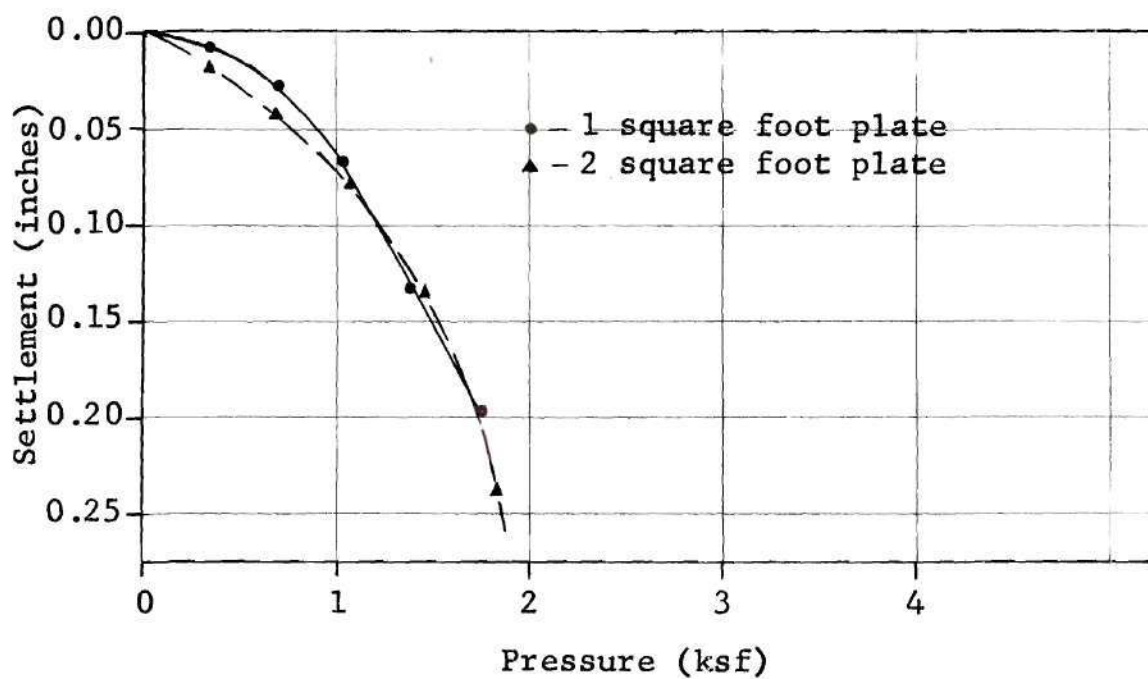


Figure 7B - Plate Settlements, Pit at 90 Per Cent Compaction - Flooded Condition

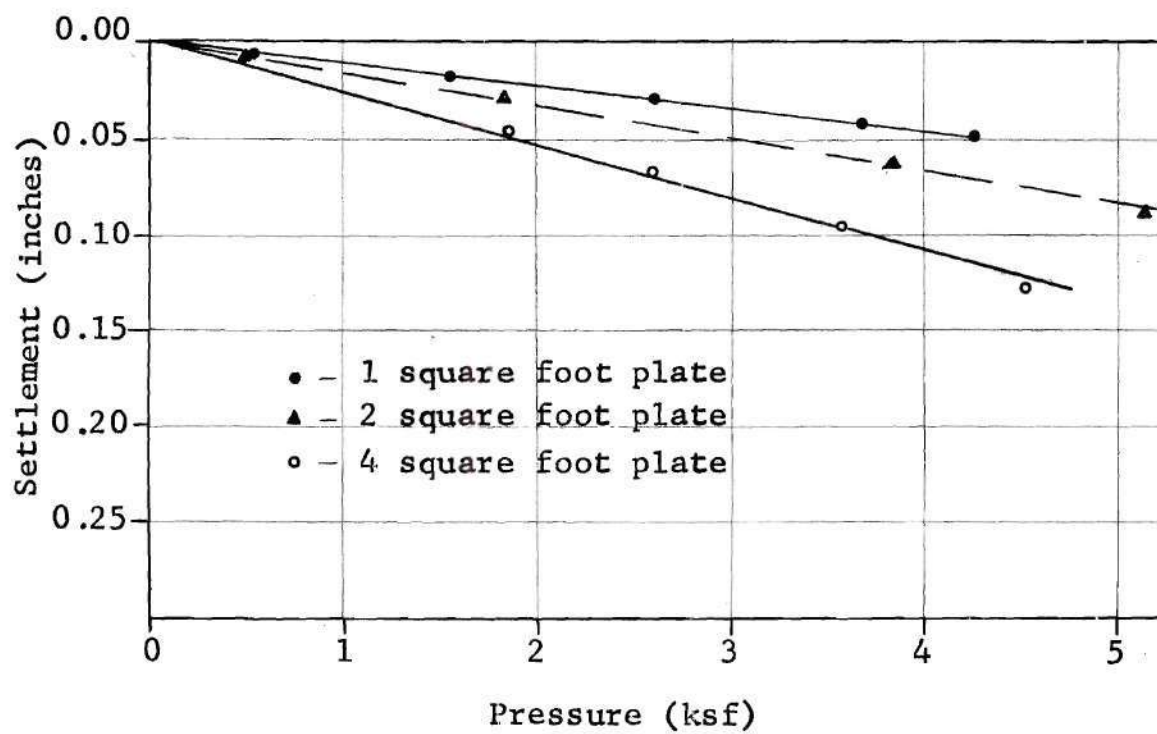


Figure 8 - Plate Settlements, Pit at 94.5 Per Cent Compaction

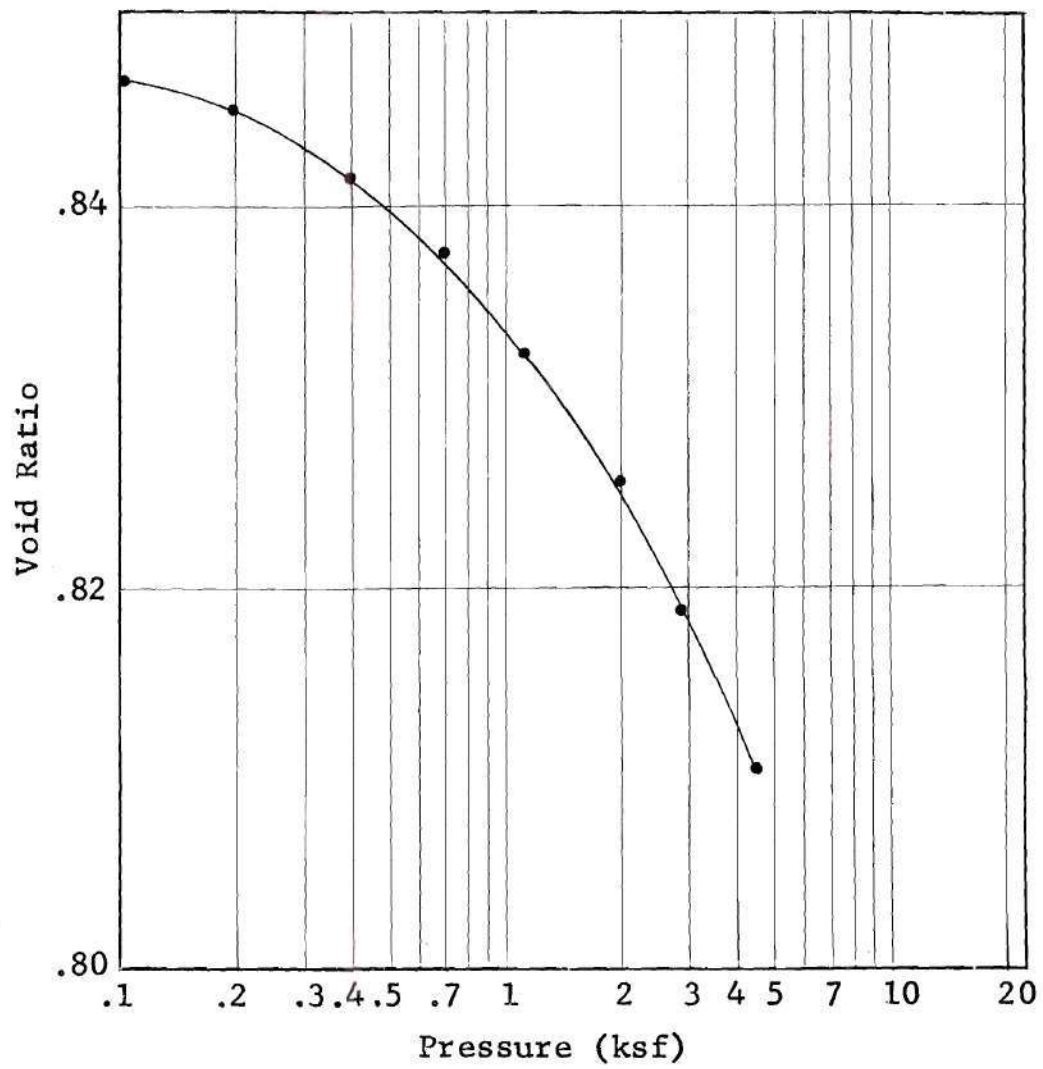


Figure 9 - Oedometer Test at 85 Per Cent Compaction

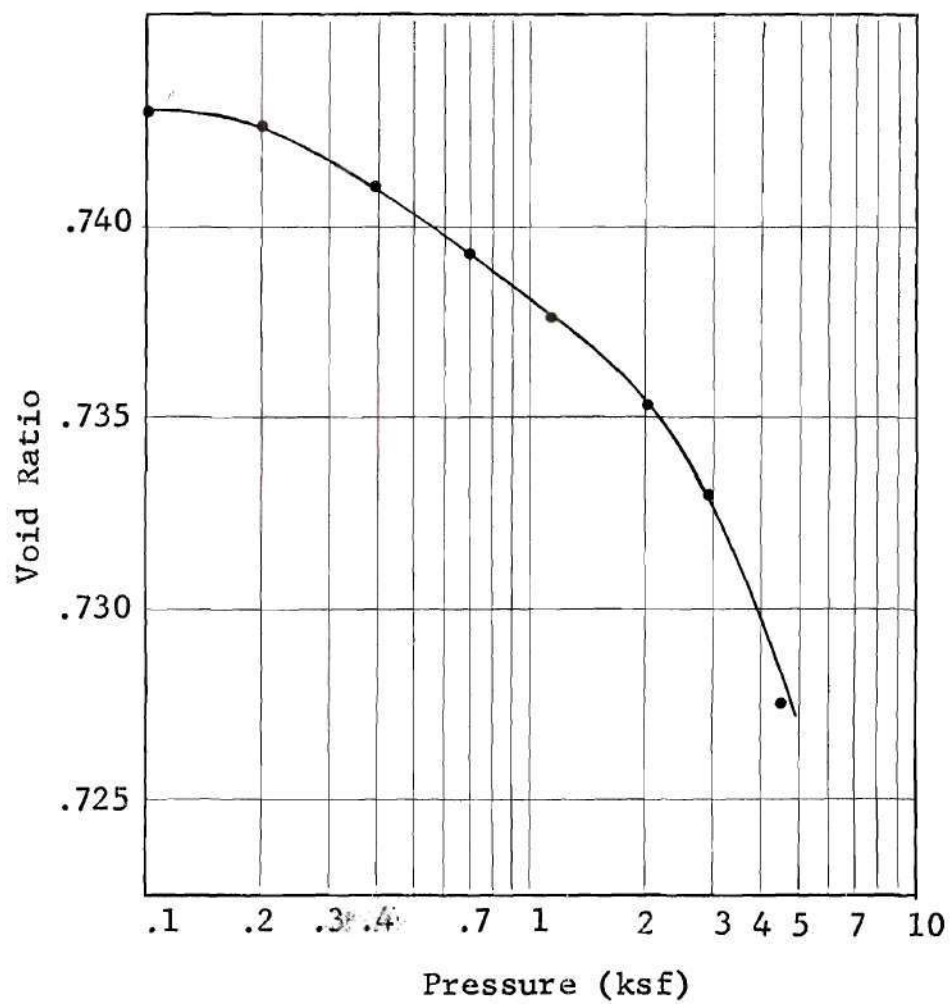
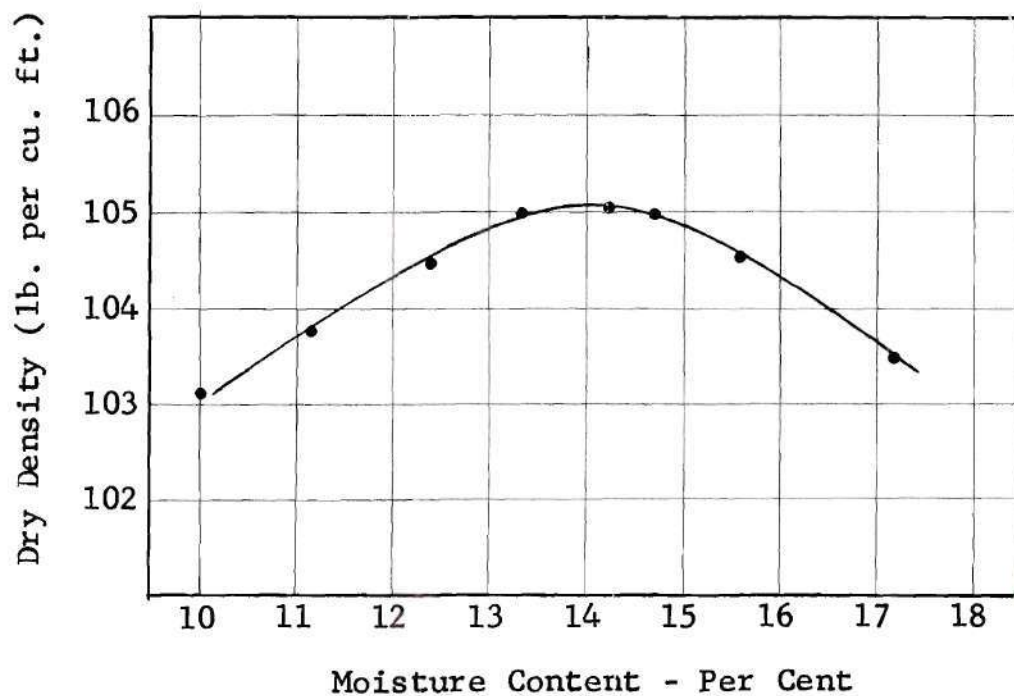


Figure 10 - Oedometer Test at 90 Per Cent Compaction



Maximum Dry Density - 105.1 lb. per cu. ft.

Optimum Moisture - 14.0 per cent

Method of Test - ASTM D-1557

Minimum Dry Density - 77.4 lb. per cu. ft.

Soil - Grey slightly silty fine sand

Figure 11 - Compaction Test Results

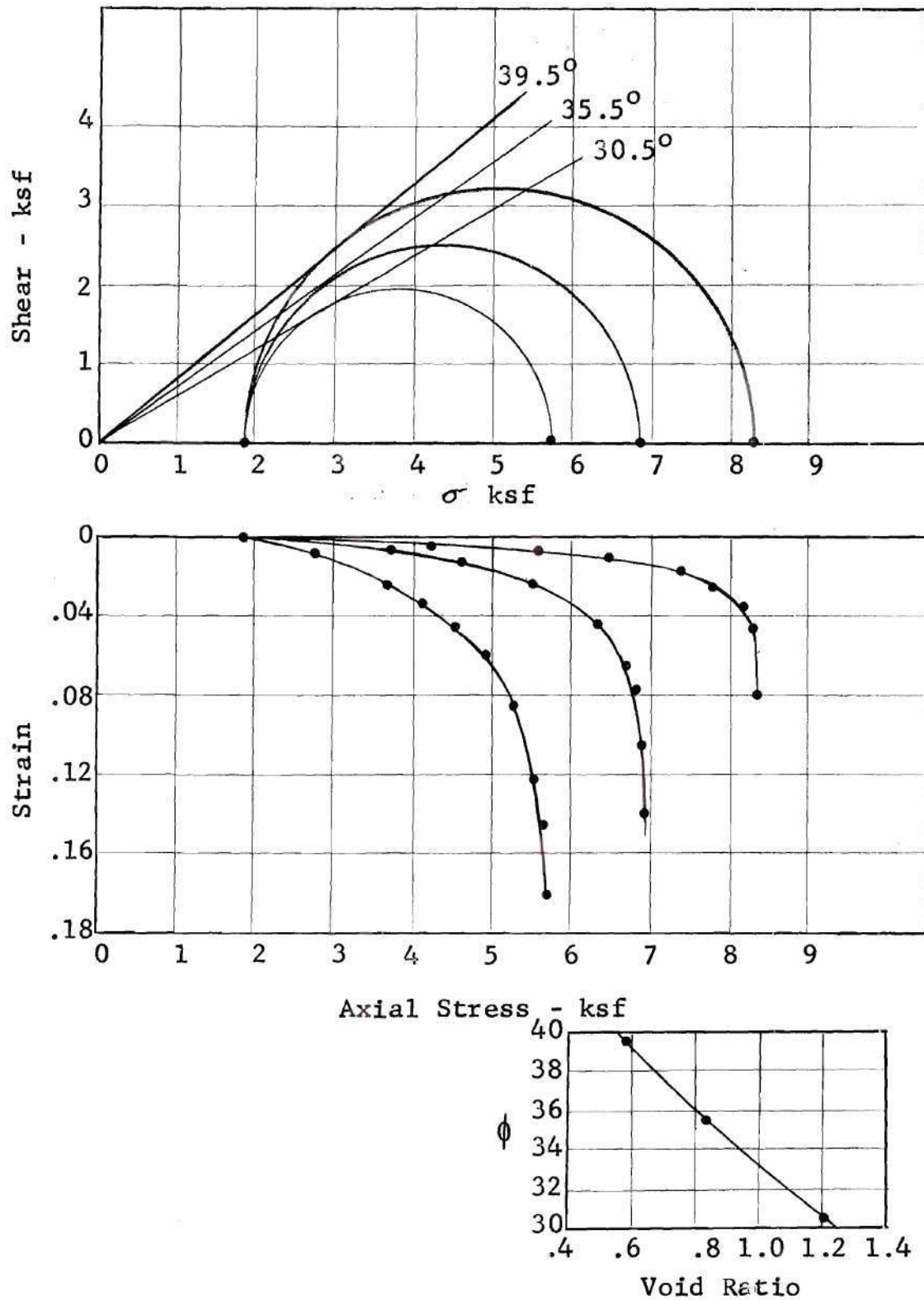


Figure 12 - Vacuum Shear Test, Dry Condition

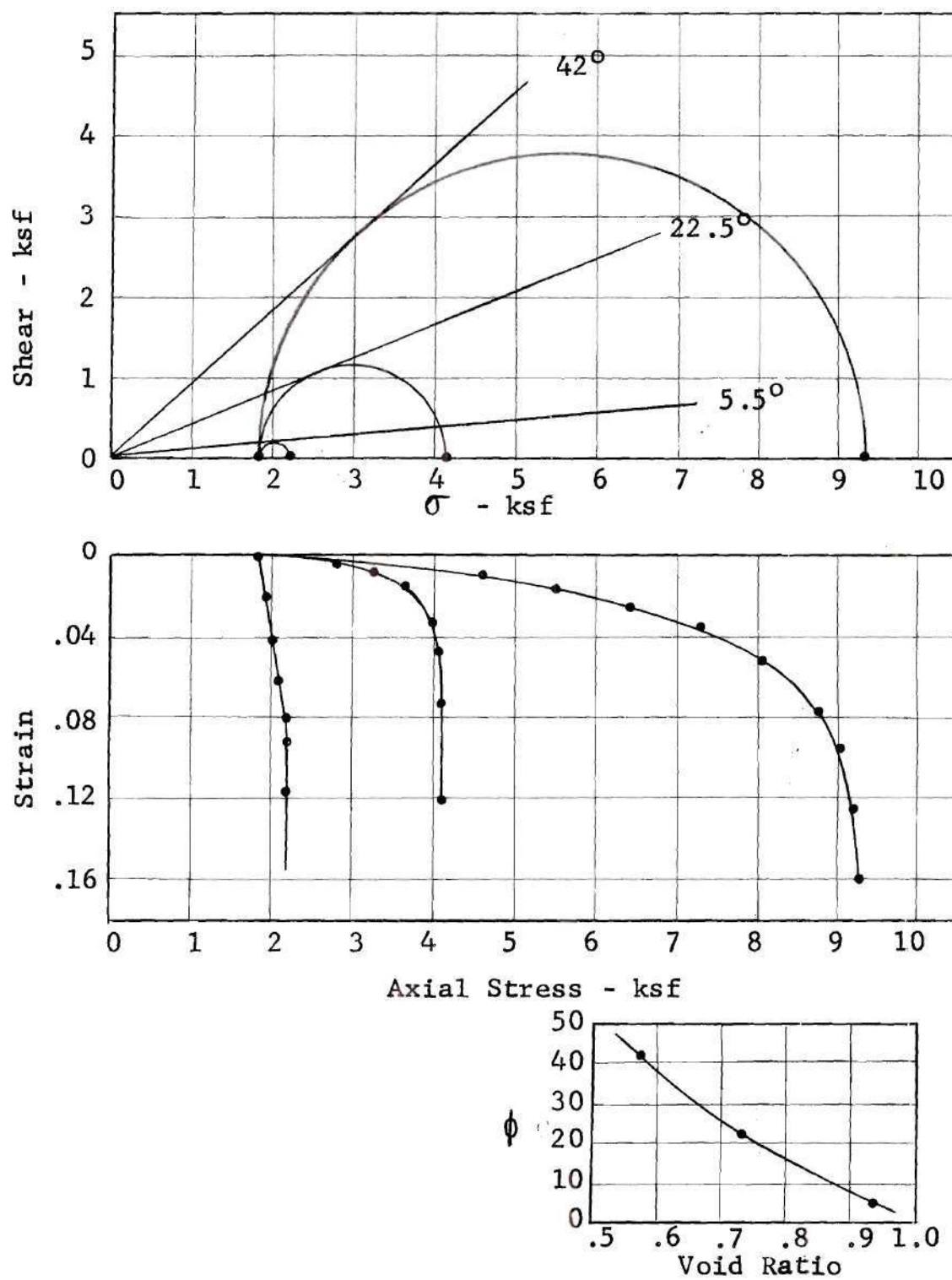
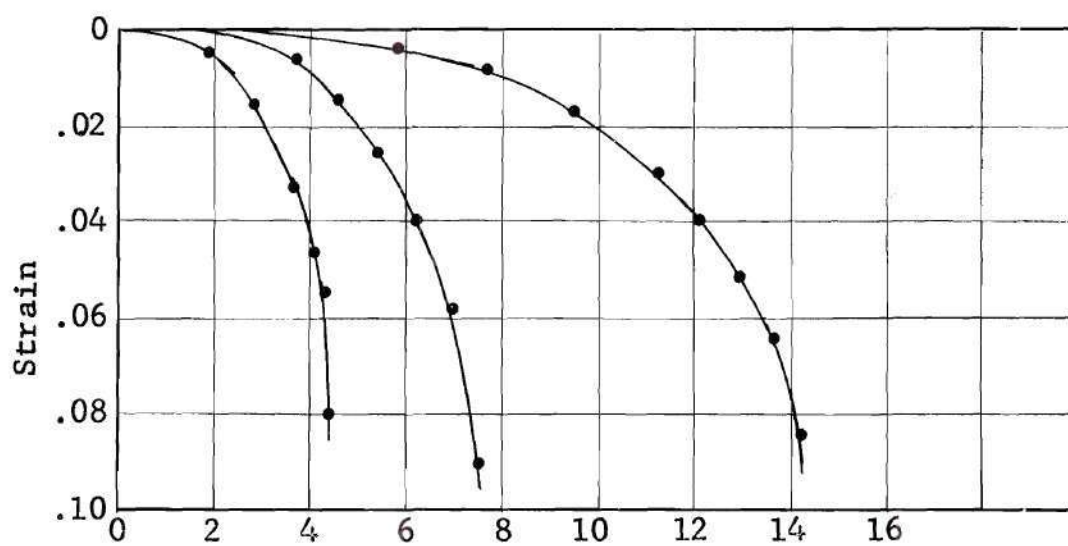
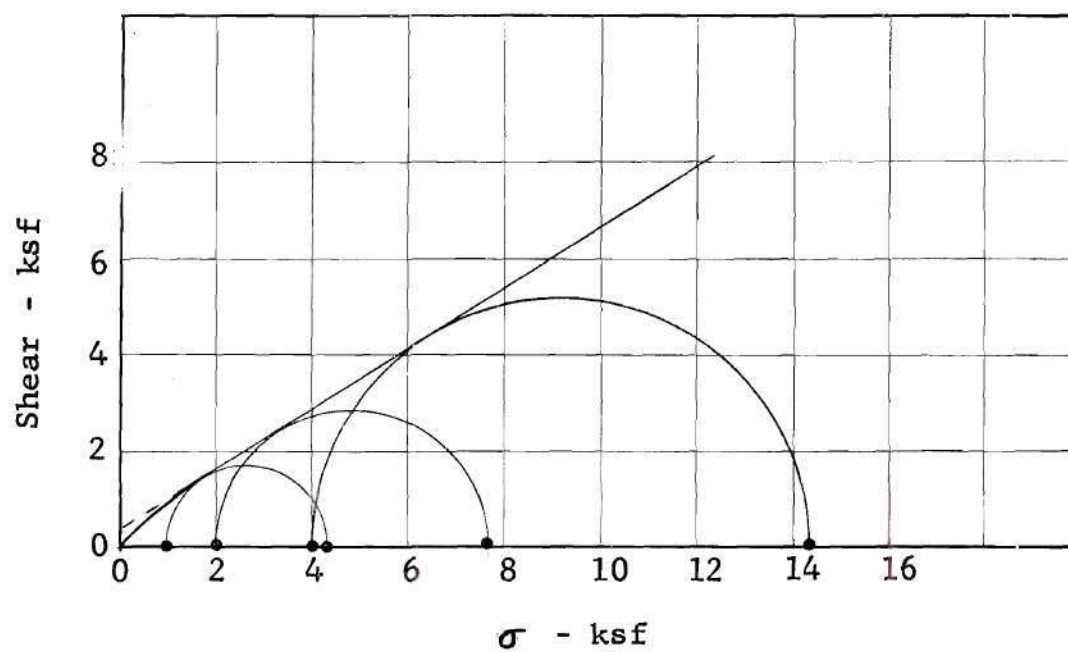
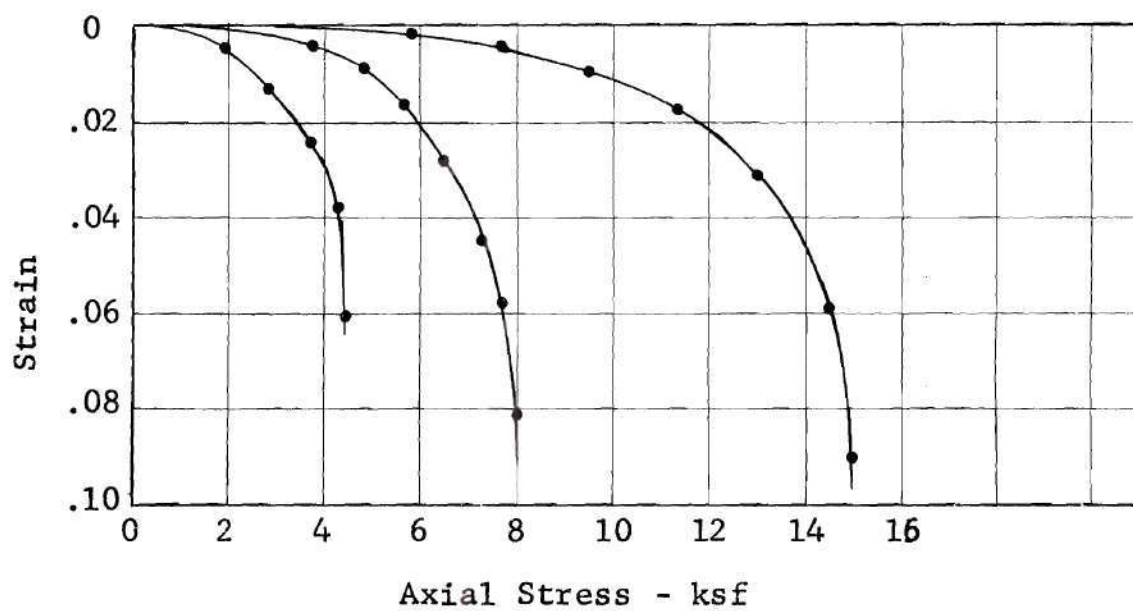
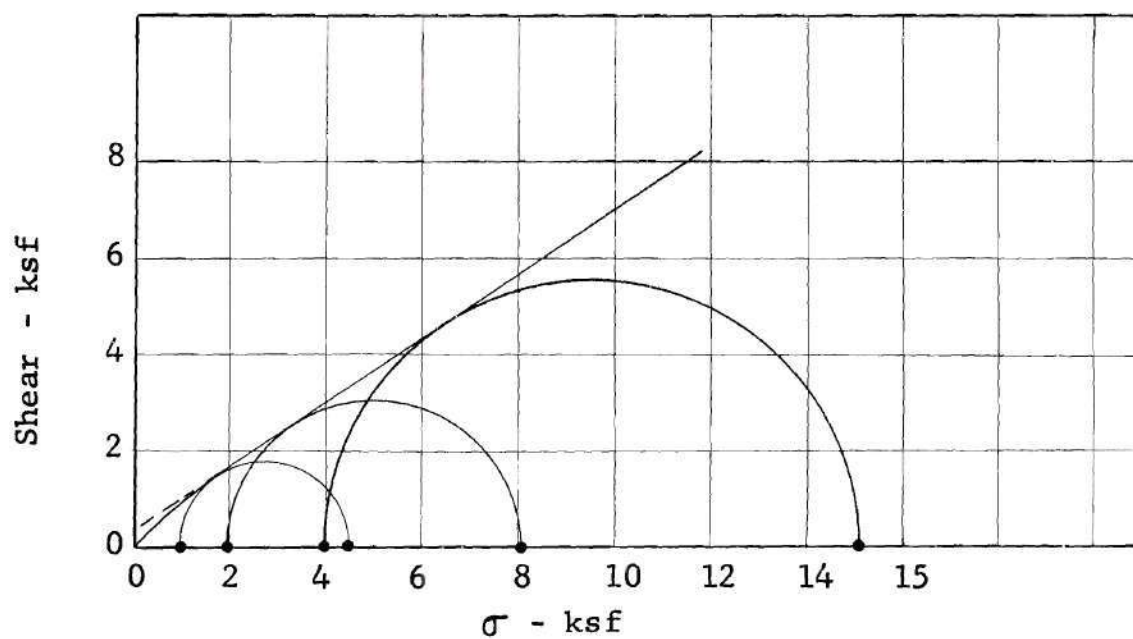


Figure 13 - Vacuum Shear Test; Saturated; Undrained



Angle of Shear Resistance, $\phi - 32.5^\circ$

Figure 14 - Triaxial Shear Test; 85 Per Cent Compaction - Consolidated, Drained



Angle of Shear Resistance $\phi - 34.0^\circ$

Figure 15 - Triaxial Shear Test; 90 Per Cent
Compaction - Consolidated, Drained

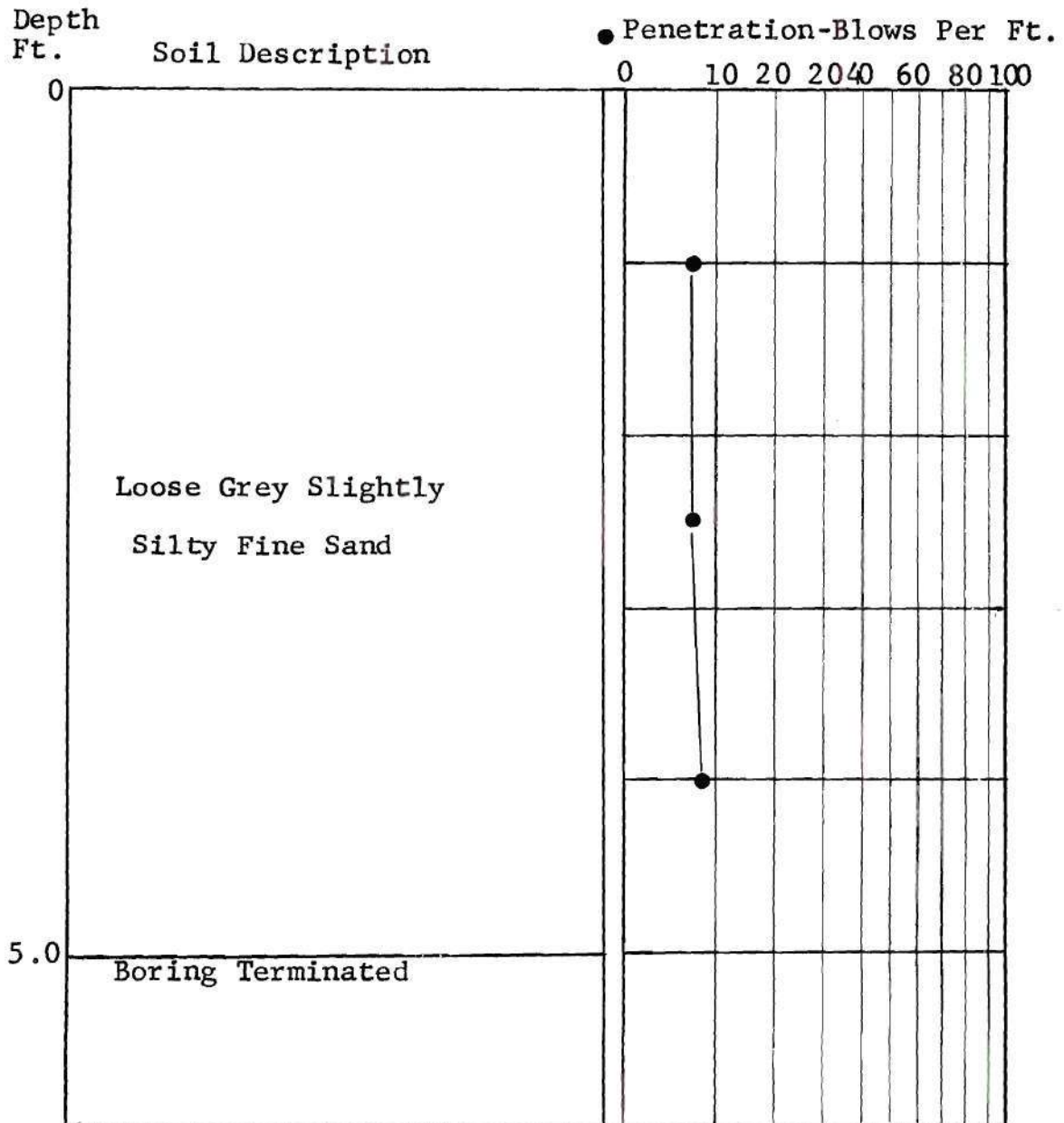


Figure 16 - Penetration Resistance of Soils at 95 Per Cent Compaction

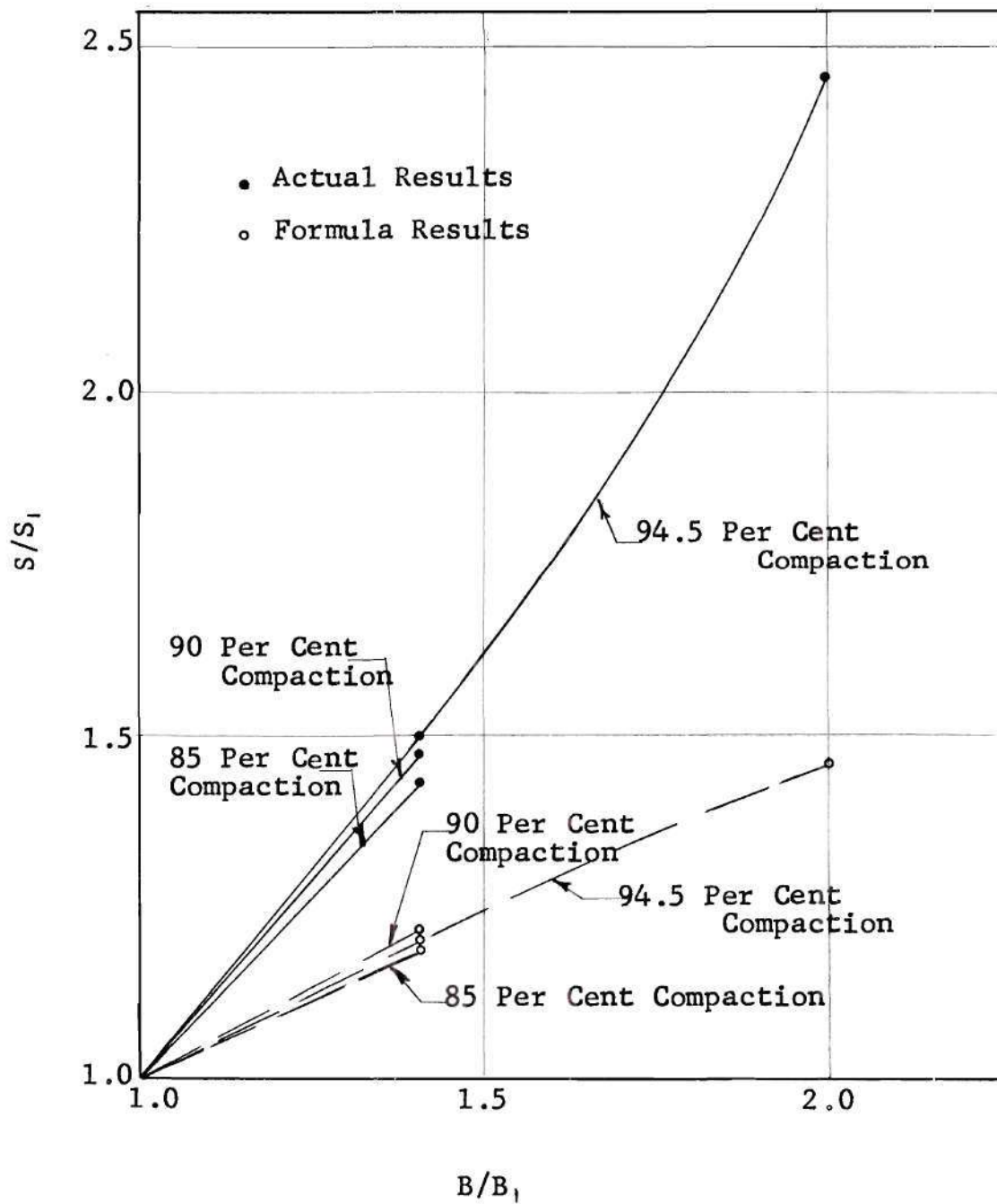


Figure 17 - Comparison of Plate Load Test Settlements to Settlements Predicted by Hough's Formula

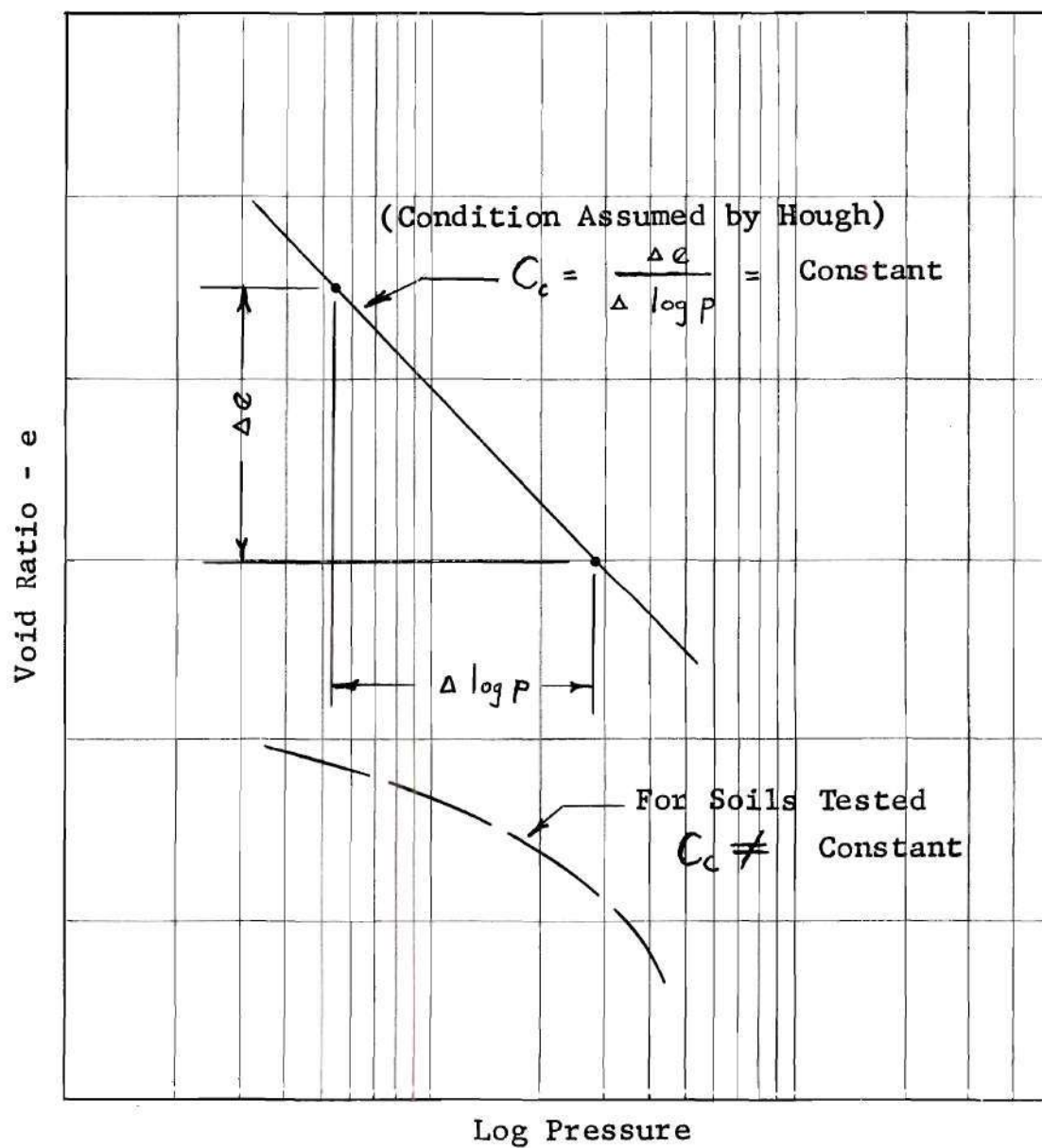
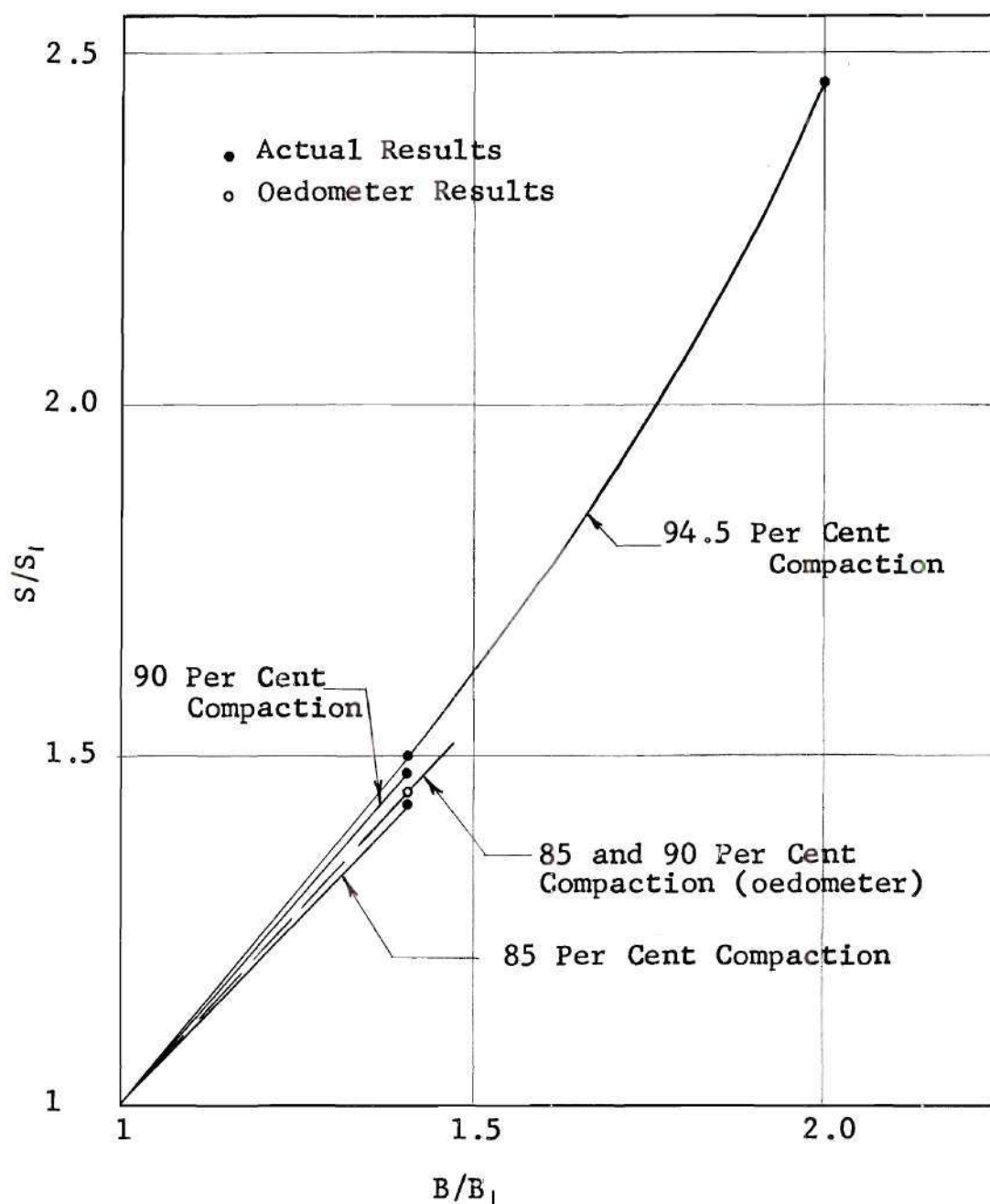


Figure 18 - Comparison of Assumed Pressure - Void Ratio Relationship to Probable Pressure - Void Ratio Relationship



Note: Although the relationships of settlement to plate size are in close agreement; the total magnitude of settlement predicted from oedometer tests was in the order of 100 per cent greater than the actual value.

Figure 19 - Comparison of Plate Load Test Settlements to Settlements Predicted by Oedometer Tests

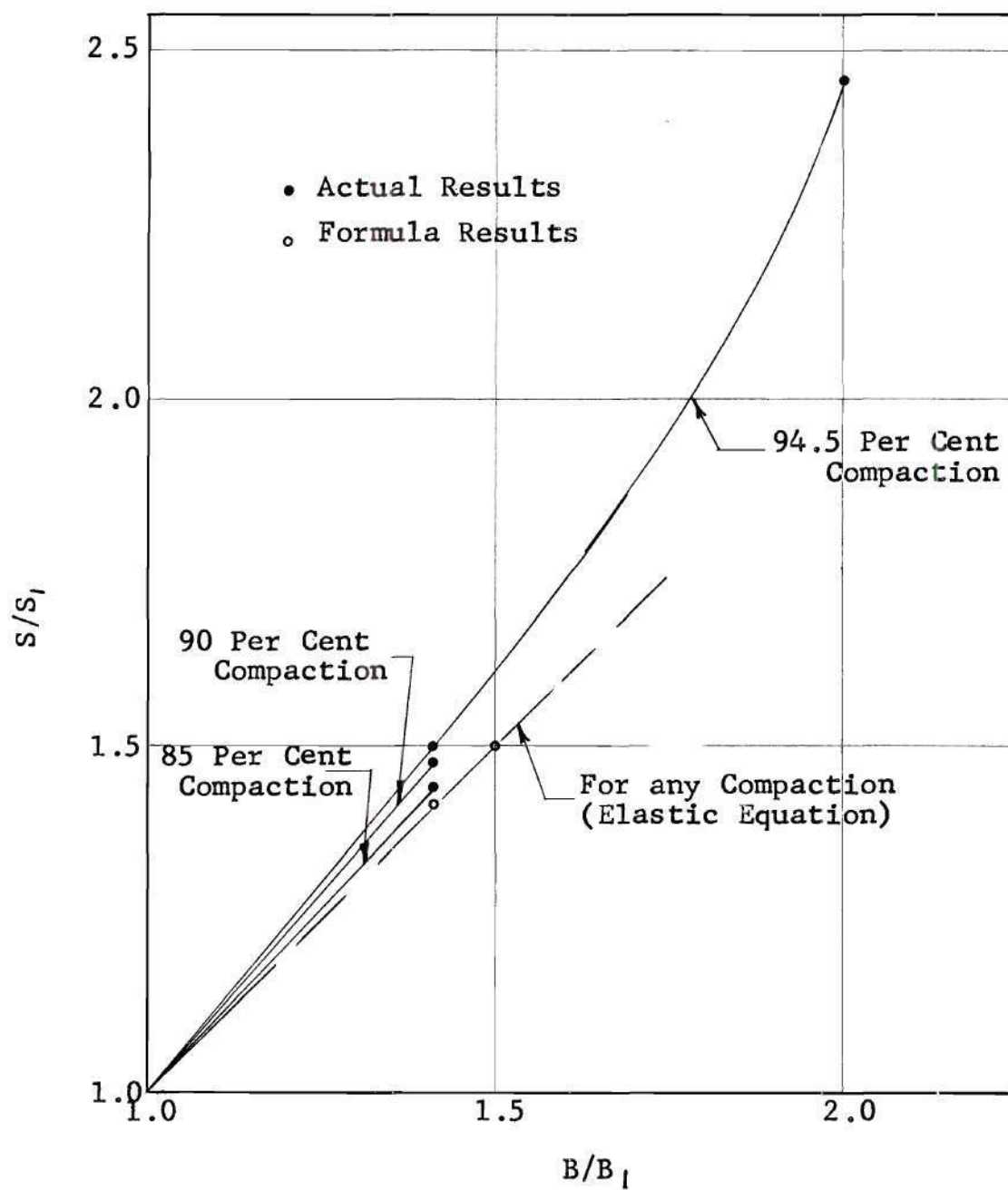


Figure 20 - Comparison of Plate Load Test Settlements to Settlements Predicted by Elastic Equation

Glossary of Terms

(Equations Developed by Hough)

Basic simultaneous equations for plates at ground surface:

$$h_s(h_s + B)^2 = \frac{10 B^2}{\gamma} p_c \quad (1)$$

$$\frac{10}{27} h_s \left(10^{\frac{SC}{h_s}} - 1 \right) (h_s + 3B)^2 = \frac{10 B^2}{\gamma} p_c \quad (2)$$

h_s = depth of significant stress

p_c = contact pressure

B = footing width

γ = effective soil unit weight

S = settlement

C = bearing capacity index (where: $C = \frac{1+e}{C_c}$)

Sample Calculation of Settlement Using Hough's Equation

Basic simultaneous equation by plates at ground surface:

$$h_s (h_s + B)^2 = \frac{10 B^2}{\gamma} p_c \quad (1)$$

$$\frac{10}{27} h_s \left(10^{\frac{SC}{h_s}} - 1 \right) (h_s + 3B)^2 = \frac{10 B^2}{\gamma} p_c \quad (2)$$

Given: Plate load test data for nominal (one square foot) plate size

Solve For: Settlements beneath larger plates for comparison to actual test results

Given: $B = 1$ foot

$\gamma = 93$ pcf

$p_c = 1000$ psf

$S = .005$ foot

Solve For: h_s , C

a) Solve Equation (1) for ' h_s ' by trial and error

$$h_s (h_s + B)^2 = \frac{10 \cdot 1}{93} = 107.5$$

Trial h_s (feet)	4	4.10	4.12
Resulting $h_s (h_s + B)^2$	100	106.9	107.6

$$107.6 \approx 107.5$$

$$\therefore h_s = 4.12 \text{ feet}$$

- b) Solve equation (2) for 'C' using 'h_s' of 4.12

$$\frac{10}{27} (4.12) \left[10^{\frac{(.005)(C)}{(4.12)} - 1} \right] [4.12 + 3]^2 = 107.5$$

$$10^{.001213 C} = 2.391$$

(from logarithmic tables)

$$C = 312$$

- c) The derived 'C' value is now substituted into the equations to determine settlements beneath the two square foot plate.

Given: B = 1.414 feet

Solve For: h_s and S

γ = 93 pcf

p_c = 1000 psf

C = 312

From equation (1); h_s = 5.09 feet (beneath 2 square foot plate)

From equation (2); S (computed) = .00593 feet

Actual Settlement beneath 2 foot plate = .00709 feet

Computed Settlement beneath 2 foot plate = .00593 feet

- d) All settlements using the two equations developed by Hough proved to be too small. The most obvious point of questionable validity is the 'C' term, Bearing Capacity Index.

Sample Calculation for Settlement from Oedometer Test Data

(Settlement Beneath 1 foot Plate at 1000 psf)

B = 1 foot
 $\gamma = 93$ pcf

Influence Value Factors are from the
 Boussinesq Equation, Average Values

Zone	z	z/B	Stress Factor	$\Delta \sigma$	σ_o	$\sigma_o + \Delta \sigma$	e_o	$e_o + \Delta e$	Δe	Δt	Inches $\sum \Delta t$
0-.25	.125	.125	.78	780	11.6	792	.8466	.8358	.0108	.0175	.0175
.25-.50	.375	.375	.69	690	34.9	725	.8466	.8368	.0098	.0159	.0334
.50-.75	.625	.625	.52	520	58.2	578	.8466	.8388	.0078	.0127	.0461
.75-1.0	.875	.875	.39	390	81.4	471	.8466	.8403	.0063	.0102	.0563
1.0-1.5	1.25	1.25	.25	250	116.2	366	.8465	.8420	.0045	.0146	.0709
1.5-2.0	1.75	1.75	.15	150	162.8	313	.8458	.8428	.0030	.0097	.0806
2.0-3.0	2.50	2.50	.07	70	232.5	302	.8442	.8431	.0011	.0071	.0877
3.0-4.0	3.50	3.50	.04	40	325.5	365	.8427	.8420	.0007	.0045	.0922

Tabulation of Actual vs Oedometer Predicted Settlements

At 85 Per Cent Compaction:

Plate Size	Actual	Computed	Per Cent	Actual
1 sq.ft.	.060	.0922	54 %	
2 sq.ft.	.085	.1332	57 %	

At 90 Per Cent Compaction:

1 sq.ft.	.033	.0693	110 %
2 sq.ft.	.048	.0993	107 %

BIBLIOGRAPHY

LITERATURE CITED

- (1) B. K. Hough, "Compressibility as the Basis for Soil Bearing Value", Journal of the Soil and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. SM4, August, 1959, pp. 11-39.
- (2) Ibid., pp. 11-39.
- (3) D. D. Barkan, Dynamics of Bases and Foundations, New York, McGraw-Hill, 1962, pp. 17-30.
- (4) George F. Sowers, Laboratory Manual for Soil Testing, Atlanta, Georgia Institute of Technology, 1962.
- (5) Karl Terzaghi and Ralph B. Peck, Soil Mechanics in Engineering Practice, New York, John Wiley and Sons, Inc. 1948, pp. 419-423.
- (6) Barkan, pp. 17-30.
- (7) Barkan, pp. 17-30.
- (8) G. A. Leonards, Foundation Engineering, New York, McGraw-Hill, 1962, p. 542.

OTHER REFERENCES

1. Lambe, T. William, Soil Testing for Engineers, New York, John Wiley and Sons, Inc., 1951.
2. Sowers, George B. and George F. Sowers, Introductory Soil Mechanics and Foundations, New York, Macmillan, 1961.
3. Terzaghi, Karl, Theoretical Soil Mechanics, London, Chapman and Hall, Ltd., 1943.